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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
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COMPACTION OF SANDS AND BEARING CAPACITY OF PILES

G. G. Meyerhof,[1] F. ASCE

SYNOPSIS

Some laboratory and field investigations of the compaction of sands are analyzed in order to obtain information on the fundamental compaction characteristics, especially under impact. A method is developed to estimate the compaction of cohesionless soils near the shaft and base of driven piles and displacement caissons of the Franki type. The theoretical results are compared with field observations.

The author's previous bearing capacity theory of piles is extended to include the effect of compaction and greater friction angles of granular soils near the shaft and base. A method is suggested for estimating the settlement and allowable load of single driven piles and displacement caissons and of groups in cohesionless soils. The theory is compared with the results of some field observations.

INTRODUCTION

The bearing capacity and settlement of piled foundations in cohesive soils can generally be estimated from bearing capacity and consolidation theories using the strength and deformation properties of representative soil samples with an allowance for changes by installing the piles. For piles driven into cohesionless soils, however, the influence of compaction on the soil properties is frequently ignored; previous methods of estimating the bearing capacity and settlement of piled foundations from static formulas are based on the assumption that the soil conditions remain unchanged by pile driving.

In loose cohesionless soils the driving of piles increases the relative density, which can be considerably raised in the case of displacement

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caissons where the base is expanded resulting in a much greater bearing capacity and smaller settlement than correspond to uncompacted materials. On the basis of some laboratory and field investigations a method is developed below to estimate the influence of the compaction of cohesionless soils on the bearing capacity and settlement of driven piles and displacement caissons.

Compaction Properties of Sands

When a pile is driven into cohesionless soil, the soil is compacted by displacement and vibration resulting in permanent rearrangement and some crushing of the particles as well as prestressing of the soil mass near the pile. A similar compaction is obtained when driving the plugged casing of a Franki displacement caisson. When the base of such a caisson is formed and expanded, the concrete is compacted by heavy blows of a drop hammer and driven out of the bottom of the casing; this concrete in turn considerably further compacts the surrounding soil. The degree of soil compaction depends mainly on the intensity and duration of the pressure pulse. Recent field observations with various types of compaction equipment have shown that for a given soil type and water content the limiting value of the dry density obtained by compaction is almost exclusively governed by the limiting value of the peak transient pressure induced in the soil.(1) In cohesionless soils the degree of compaction was found to be practically independent of the pressure pulse duration, but it was enhanced by continuous vibration roughly in proportion to the additional local pressures resulting from oscillation.

Since the degree of compaction of cohesionless soils near driven piles and the base of displacement caissons is governed by the impact pressures induced in the soil under the energy of the hammer, some laboratory tests were made to provide information on the relationship between the energy per blow and the compaction of loose sand. This investigation, which is reported in detail elsewhere,(2) consisted of dropping weights of 3.8 in. diameter and varying from 5 to 20 lb. from different heights of 1 to 24 in. on the surface of loose medium Ottawa sand at various moisture contents in a 4 in. dia. container. The thickness of the soil layers varied from about 2 to 6 in. and the number of blows per layer varied from 1 to 25. After each test the resulting density of the sand was determined. In addition static confined compression tests were made.

For the purpose of the present analysis the peak pressures in the impact tests may be estimated by equating the energy per blow (immediately after impact of the drop hammer) to the work done in deforming the sample. Thus, on the assumption of an approximately parabolic pressure-density relationship,

$$Wh \times \frac{W}{W + P/2} = pAs/3 \qquad (1)$$

or the peak pressure

$$p = \frac{3Wh}{As} \times \frac{W}{W + P/2} \qquad (2)$$

where W = weight of hammer,

P = weight of sample,

A = area of sample,

h = height of fall of hammer

and s = permanent compression of sample.

Typical experimental results (Fig. 1) show that the dry density of the soil increases with the peak pressure at a decreasing rate, and generally only a small increase was found beyond a pressure of about 50 lb./sq. in. The pressure-density relationships are independent of the weight of the hammer; for weights exceeding those used in the tests and under a great number of blows, more crushing of the soil grains would however probably produce somewhat greater densities. At a given peak pressure and thickness of the layer, the density increases with the number of blows per layer at a decreasing rate, and no significant increase seems to occur beyond about 10 blows. The densities are somewhat greater for the thinner layers, which may be explained by the smaller side friction on the container and the greater average pressures compared with those for the thicker layers. Confined compression tests showed that impact pressures produce greater densities than corresponding static pressures when the restraint by arching is much greater (Fig. 1). Similar results have been obtained in impact compaction tests of moist and fully saturated sands. For sand at the optimum moisture content the degree of compaction was found to be greater than for dry and fully saturated sands under the same peak pressure, as is the case in Proctor compaction tests.

Further data on the pressure-density relationships of sands under various methods of compaction can be obtained from an analysis of some field observations on moist material.[1,3] The results are shown in Fig. 2 in terms of the applied peak pressure and measured relative density of the sands together with some previous static tests[4] and the present impact tests on dry sand in the laboratory. Static pressures give the smallest and intense vibration the greatest degree of compaction, while impact pressures cause only a

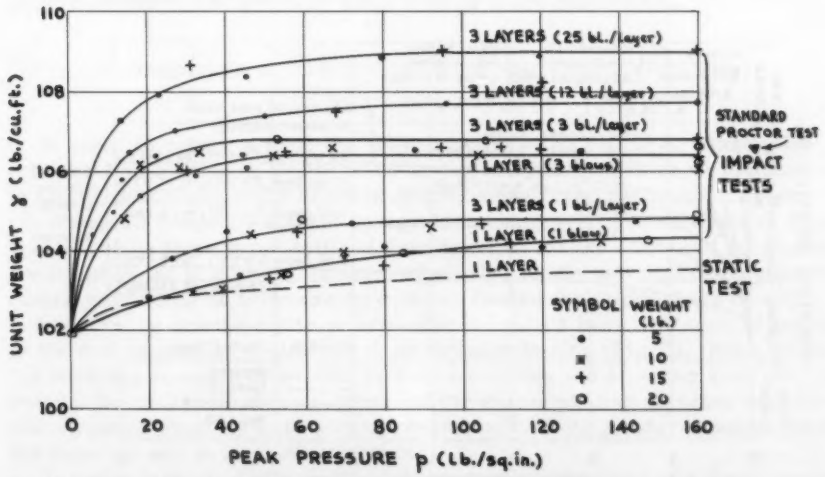

FIG. 1. RESULTS OF COMPACTION TESTS ON DRY OTTAWA SAND

moderate amount of vibration and like rolling produce an intermediate degree of compaction. It has also been found that more than one-half of the final compaction is obtained under 1 blow or pass of the compactor, while beyond about 5 blows under constant impact or about 10 passes under rolling comparatively little further compaction is obtained. The increase in relative density under a given pressure change is greater for loose soils than for dense soils, as would be expected from their difference in compressibility. In all cases the compaction increases with peak pressure at a decreasing rate, and little further compaction occurs beyond a pressure of about 3 or 4 t./sq. ft. for the sands tested.

The general experimental results illustrated in Fig. 2 by the relative density-pressure relationships are similar to the corresponding void ratio-pressure curves obtained in static confined compression tests. Static void ratio-pressure curves are frequently represented by(5)

$$e = e_1 - C_c \log_{10}(p/p_c + 1) \qquad (3)$$

where e_1 = initial void ratio,

p = applied effective pressure,

p_c = pressure constant

and C_c = compression index.

Since for cohesionless soils the void ratio approaches a final value e_2 for large pressures, Eq. (3) is conveniently replaced by an analogous relationship

$$e = e_2 + (e_1 - e_2)10^{-(p/p_c)^C} \qquad (4)$$

where C = compaction index, which can be determined from the void ratio-pressure curves similar to the compression index.

After substituting into Eq. (4) the following equation for relative density of the soil

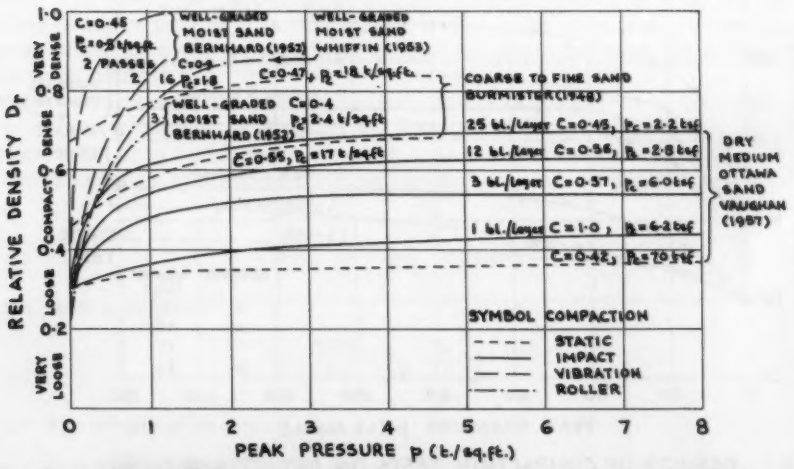

FIG. 2. EXPERIMENTAL DENSITY-PRESSURE CURVES FOR SANDS

$$D_r = (e_o - e)/(e_o - e_m) \quad (5)$$

where e_o = void ratio in loosest state of packing

and e_m = void ratio in densest state of packing,

one obtains

$$D_r = D_2 - (D_2 - D_1)10^{-(p/p_c)C} \quad (6)$$

where D_1 and D_2 are the relative densities at e_1 and e_2, respectively. Since

$$10^{-(p/p_c)C} = \frac{1}{1 + 2.3(p/p_c)^C}$$

approximately, Eq. (6) becomes, approximately,

$$D_r = D_2 - \frac{D2 - D1}{1 + 2.3(p/p_c)^C} \quad (6a)$$

The experimental relationships between relative density and peak pressure can readily be expressed by Eq. (6) and the values of the corresponding compaction index C and pressure constant p_c are given on the various curves of Fig. 2. The compaction index is found to lie between about 0.4 and 1.0; the pressure constant varies from about 0.5 to 70 t./sq. ft., the lower values applying to vibration, intermediate values to impact or rolling and the upper values to static compaction.

For fully saturated sands the pressures for a given degree of compaction are greater than for a dry material on account of the dynamic porewater pressures induced, especially in fine sands. For coarse cohesionless soils, however, the pressure-density relationships of dry materials can probably be used in conjunction with the effective peak pressures. In addition, the physical characteristics of the sand affect the degree of compaction considerably. When more data become available, it should be possible to relate the compaction properties of granular soils to their index properties.

Compaction of Cohesionless Soils near Driven Piles and Displacement Caissons

In order to estimate from the above equations the degree of compaction of cohesionless soils near driven piles or displacement caissons, it is necessary to calculate the magnitude of the peak pressures at the base and in the surrounding soil under the applied energy of the hammer. The analysis is first made for a uniform loose soil and then modified for the effects of compaction produced by the pile or the caisson, of which a single unit will be considered before suggesting an extension to groups. During the initial stage of soil compaction by driving a pile or expanding a caisson base, the state of stress in the soil is similar to that of a deep circular footing (Fig. 3). Near the base the shearing strength of the soil is fully mobilized and a plastic zone develops, for which the shear pattern and the corresponding stresses and ultimate bearing capacity were given previously.(6,7) At greater distance from the base the soil is in an elastic state.

In order to drive a pile or expel concrete from the base of a displacement caisson and produce compaction of the soil, the stresses induced in the soil

must be of such magnitude that substantial permanent deformations are caused. The energy per blow of the hammer must, therefore, be great enough to overcome the ultimate bearing capacity of the soil near the base. At installation of a pile or expanding a caisson base, the required minimum force is thus the sum of base resistance and skin friction or

$$R = q\,A + fS \tag{7}$$

where q = unit base resistance,

f = average unit shaft friction,

A = area of base

and S = surface area of shaft.

In cohesionless soils dynamic formulas give reasonably accurate results, especially in the case of a bulbous base when the mass struck is compact. A great accuracy of the estimate of the resistance is not required because the degree of compaction increases approximately with the logarithm of the applied pressure (see Fig. 2 and Eq. (6). The probable value of the dynamic resistance may thus be deduced from the Hiley formula,[8] which gives

$$R = \frac{\eta\, Wh}{s + c/2} \tag{8}$$

where η = efficiency of blow,

W = weight of hammer,

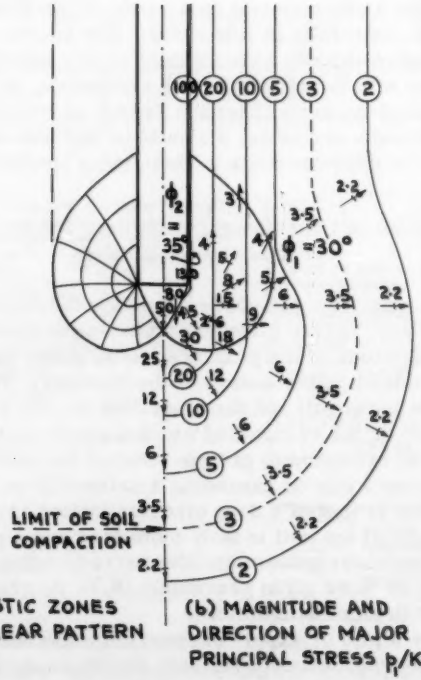

FIG. 3. PLASTIC ZONES AND MAJOR PRINCIPAL STRESS AFTER DRIVING CIRCULAR PILE

h = height of free fall of hammer,

s = penetration (set) of base per blow

and c = temporary elastic compression of pile or base and soil.

Substituting Eq. (8) into (7) the minimum required energy per blow of the hammer during the pile driving or expansion of the caisson base is, approximately,

$$Wh = (qA + fS)(s + c/2)/\eta \tag{9}$$

The major principal stress induced by the dynamic force given by Eq. (8) is taken as the pressure producing compaction of the soil. The magnitude and direction of this pressure can be calculated from plastic theory within the failure surface near the base and elastic theory at greater distance from the base. In the plastic zones of a deep circular pier in cohesionless soil the major principal stress is found to be very nearly given by[6]

$$p_1 = \lambda K \gamma D e^{2\theta \tan\phi} \cdot \tan^2 (45^\circ + \phi/2) \tag{10}$$

where γ = unit weight of soil,

θ = angle between vertical and direction of stress,

λ = shape factor,

ϕ = reduced angle of internal friction allowing for compressibility of the soil,

D = depth of base

and K = coefficient of earth pressure.

In the elastic zone the major principal stress is determined from Boussinesq-Mindlin equations[8] using the principal stresses calculated from Eq. (10) and applied along the failure surface; this stress can be expressed by

$$p_1 = \frac{3qB^2}{16r^2} \cos\theta \tag{11}$$

where B = width of base,

r = distance from footing centre

and the base pressure q is a function of the same parameters as used in Eq. (10).

In the plastic zones the major principal stress follows a path of the bisector of the angle between radial and tangential slip lines, while in the elastic zone this stress acts radially from the centre of the base (Figs. 3 and 4). Using Eqs. (10) and (11) the major principal stress can be computed at various points in the soil, and its value p_1 is substituted for the pressure p in Eq. (6) to give the relative density D_r of the material at the particular point. The angle of internal friction ϕ corresponding to this computed value of D_r can be ascertained from laboratory drained shear tests on representative soil samples obtained from the site. If such test results are not available, approximate values of ϕ and D_r can be deduced from the results of standard or static penetration tests,[9] from which it has been found that for sands, approximately,

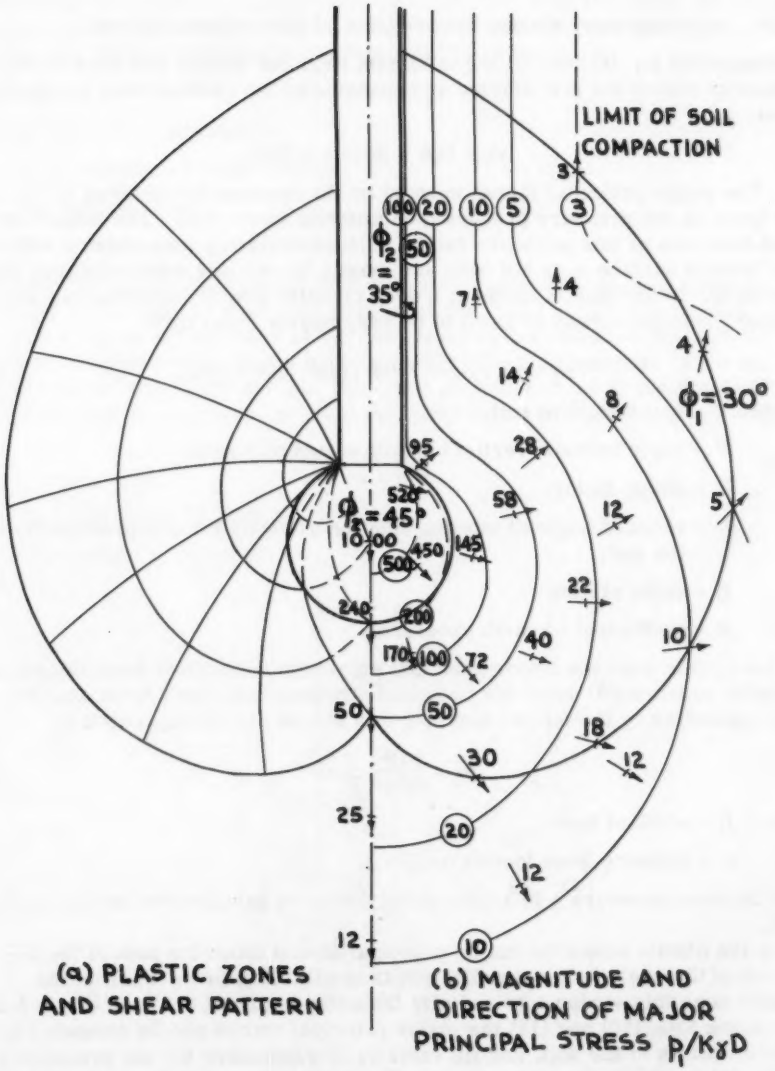

FIG. 4. PLASTIC ZONES AND MAJOR PRINCIPAL STRESS AFTER EXPANDING BASE OF CAISSON

$$\phi = 28^{\circ} + 15^{\circ} D_r \qquad (12)$$

Using the new values of ϕ, the major principal stresses are recalculated to give a new set of relative densities and revised values of ϕ. This process of iteration is repeated until the final stresses p_1 correspond to the final angles ϕ.

The results of such a computation are shown in Fig. 3 in terms of the major principal stress ratio $p_1/K\gamma D$ for a circular pile driven into loose sand with an original angle of internal friction $\phi_1 = 30^{\circ}$ and a final angle $\phi_2 = 35^{\circ}$ at the shaft and base. In any given case the major principal stress at any point can then be determined by multiplying by $K\gamma D$ the ratios given in Fig. 3. Since during pile driving the stresses in the soil around the shaft are similar to those in the plastic zones near the base, the maximum principal stresses in these zones can be projected upwards. The corresponding stress distribution is indicated in the upper part of Fig. 3 and shows that the stresses increase rapidly towards the shaft where the compaction reaches a maximum. Near the base the stresses are a maximum immediately below the base and decrease radially and tangentially in the plastic zones and radially in the elastic zone.

Fig. 3 may also be used to estimate the stresses and compaction produced by driving the casing of a displacement caisson. However during expansion of the caisson base, roughly in the direction of the major principal stresses, the plastic zones increase in proportion to the size of the base. The corresponding shear pattern is shown in Fig. 4 for the stage when the base diameter is about twice the shaft diameter. As before, the energy per blow of the hammer must be great enough to overcome the corresponding bearing capacity of the soil so that the minimum required energy per blow is given by Eq. (9) for the new dimensions of the base. Using the above iterative procedure, the magnitude and direction of the major principal stress ratio $p_1/K\gamma D$ at various points in the soil have been computed from Eqs. (10) and (11) and are shown in Fig. 4 on the assumption of a loose sand with an original angle of internal friction of $\phi_1 = 30^{\circ}$. A final angle $\phi_2 = 35^{\circ}$ has been taken along the shaft, while $\phi_2 = 45^{\circ}$ has been used for both the concrete of the base and the very dense sand near the bottom of the shaft for which the plastic zones and shear pattern are also indicated. With greater distance from the base the angle of internal friction decreases, and in the elastic zone the angle ϕ remains sensibly unchanged so that $\phi_1 = 30^{\circ}$ at the outer zone of plastic equilibrium. For intermediate plastic zones the angle ϕ has been taken to vary linearly with distance from the top of the expanded base where the impact is applied. In the upper part of the plastic zones near the shaft the maximum principal stresses are governed by the installation of the shaft and have been obtained as indicated above for a driven pile. High stresses are induced in the soil around the base and are a maximum at the bottom of the shaft; the stresses decrease roughly radially with distance from the bottom of the expanded base both in the plastic and elastic zones (Fig. 4).

It is of interest to note that the zones of equal principal stress ratios are greater below the base than above it, as would be expected from the downward and outward displacement of the soil from the base. The theoretical limits of the zone of soil compaction are at the points where the major principal stress ratio is equal to the coefficient of passive earth pressure (equal to 3 in the present cases). These limits are indicated in Figs. 3 and 4 and show that for loose sand the horizontal extent of the compacted zone along the shaft has an

overall width of about 6 times the shaft diameter increasing to about 7 or 8 times the base diameter at a short distance below the base; the compacted zone is also found to extend below the base to a depth of about 5 times the base diameter. At the outer limits of the zone of soil compaction, the major principal stress is

$$p_1 = K_o \gamma D \tan^2(45^\circ + \phi/2) \qquad (13)$$

where K_o = coefficient of earth pressure at rest and other symbols as before. Due to prestressing of the soil by compaction, the coefficient of earth pressure increases from K_o at great distance from the shaft and base to a maximum value at the pile or caisson base.

When driven piles or displacement caissons are installed in groups, the pressures induced by the individual units in the surrounding soil combine and the compaction of the material is correspondingly increased. Overlapping of the individual compaction zones occurs theoretically whenever the spacing of individual piles or caissons is less than about 7 or 8 times the base diameter (Figs. 3 and 4). In the elastic zones of the soil the total stresses can be determined by superposition of the stresses due to the individual units in order to estimate the resultant major principal stresses. The state of equilibrium in the plastic zones, however, has to be determined separately for each individual case because the shear pattern is changed considerably when the spacing of the piles or caissons is of the same order or smaller than the overall width of the failure surface of individual units, i.e., about 4 times the base diameter (Figs. 3 and 4). In the absence of a solution of this difficult problem, it is believed to be safe and not unduly conservative to use the same major principal stresses in the plastic zones as for individual piles and caissons. The degree of compaction of the soil and resulting angle of internal friction are then computed by the iterative procedure outlined above. It may be noted that the upper limit of soil compaction is given by the state of packing at the maximum relative density, which is obtained when the volume of the piles or caissons in the group is equal to that corresponding to the difference between the original and maximum soil densities within the group.

Unfortunately only a few limited field measurements of the compaction obtained at various distances from single piles driven into sands are available for comparison with the proposed method and no similar observations appear to have been published for pile groups. Some static cone penetration tests before and after pile driving have been made[10] below the point of 16 in. dia. precast concrete piles driven into submerged compact (medium dense) fine to medium sand with an original static cone resistance of about 100 t./ft.^{2}. At the pile point the cone resistance was increased by about 2 to 3 times the original value, and the depth of the compacted zone was up to 4 times the pile diameter; these ratios seemed to decrease as the original cone resistance increased. In a more extensive series of tests[11] the static cone resistance before and after driving was measured below and adjacent to 21 in. square precast concrete piles driven into submerged loose to compact fine to medium sand with an original cone resistance varying from about 50 to 150 t./ft.^{2}. At the pile point the final cone resistance varied from about 2 to 5 times the original value the difference decreasing rapidly with depth, and the depth of the compacted zone varied from about 4 to 5 times the pile width; again the upper ratios were found with the originally looser packing and the lower ratios with the originally more compact state of the sand (Fig. 5). At a lateral distance of about twice the pile width the original cone resistance was

increased by about 50 to 100 per cent, indicating that along the shaft the cone resistance is likely to have been more than doubled.

The above observations give some support to the theoretical limits of the zone of soil compaction for loose sand (Fig. 5). Since the compaction properties of the sand at this latter site are not known, however, it is only possible to compare the observed cone resistance with that estimated from the theoretical major principal stress ratios and the probable pressure-density relationship of the soil. Thus in general, the penetration resistance increases approximately parabolically with the relative density of sands[9] and since the peak pressure producing compaction also increases roughly parabolically with relative density (Figs. 1 and 2), the penetration resistance at any given point in the soil would theoretically be expected to vary in a similar way as the major principal stress at that point. The ratios of the observed final to original cone resistance are therefore shown in Fig. 5 for comparison with the theoretical principal stress ratios for an originally loose sand, and the

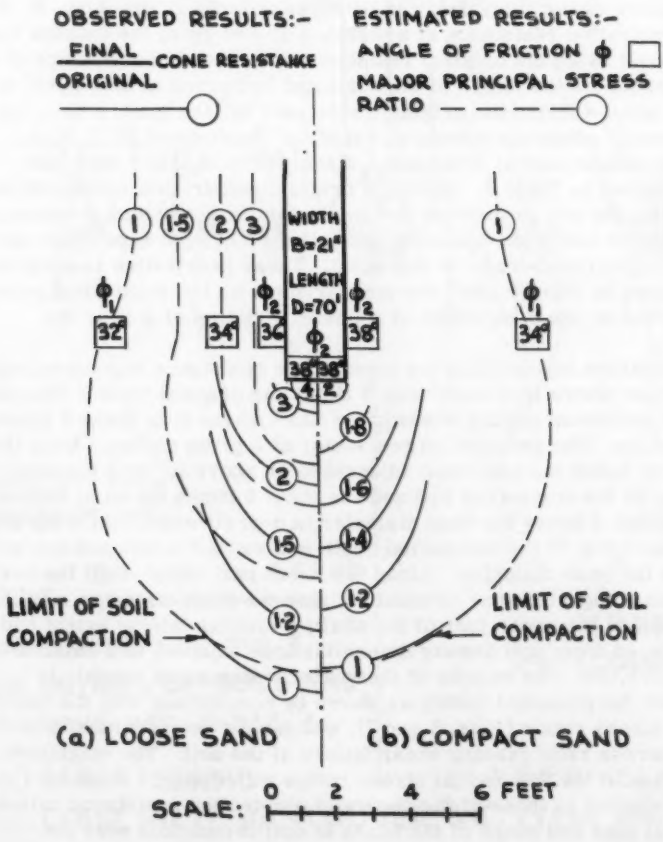

FIG. 5. COMPACTION OF SAND NEAR DRIVEN PILES

observed variation with distance from the base is found to be in reasonable agreement with the estimates.

No field data appear to be available on the extent of the zones with large permanent soil deformations near driven piles, but laboratory tests[12] indicated an overall width of about 4 times the pile diameter. Some indirect field information on the vertical extent of these zones can be obtained from the observations that good agreement between actual and predicted point resistance of driven piles in sands can be obtained by using the average static cone resistance within a depth of about 1 pile diameter below the base and about 4 pile diameters above the base.[13] These limits are of the same order as the theoretical extent of the plastic zones[7] (Fig. 3).

A check of the proposed method of estimating the compaction of sands near displacement caissons can be obtained from an analysis of the penetration tests made at about 2 ft. intervals and various distances from 2 to 12 ft. from single Franki caissons on 3 sites underlain by submerged loose medium sand with an average original standard penetration resistance of 6 to 7 blows/ft. Apart from some exploratory dynamic penetration tests at Brooklyn, N. Y., where the penetration resistance at a distance of 3 ft. from the caisson was found to be about twice the original resistance (measured at a distance of 9 ft. from the caisson) for the length of the shaft and increased at base level to a maximum of about 4 times the original resistance for the base, a detailed series of dynamic penetration tests was made at Vancouver, B. C., Lulu Island and Sea Island, and at Worcester, Mass. Details of the soils and caissons are given in Table 2. Since the original penetration resistance at a site varies with the soil properties and depth, the final resistance measured at any point in the soil after installing the caisson has been expressed as a ratio of the original resistance at that point. These penetration resistance ratios are shown in Figs. 6 and 7 for comparison with the theoretical principal stress ratios on the assumption of a linear variation of ϕ near the caisson.

The observations indicate that the penetration resistance increases rapidly towards the base where it is more than 8 times the original value. The resistance also increases rapidly towards the shaft where it is about 3 times the original value. The penetration resistance at a given distance from the base is greater below the base than adjacent to or above it. For the deep caissons (Fig. 6) the compacted soil zone is about 6 times the base diameter in depth and about 8 times the base diameter in overall width, while for the shallow caisson (Fig. 7) the compacted depth is about 3.5 times and the width about 5 times the base diameter. Along the lower part of the shaft the overall width of the compacted zone is about 7 times the shaft diameter, and this width decreases in the upper part of the shaft; a similar lateral extent had also been deduced from soil density determinations adjacent to 2 caisson shafts elsewhere.[14] The results of these observations are roughly in agreement with the proposed theory as shown by comparison with the theoretical principal stress ratios (Figs. 6 and 7), which have been expressed in terms of the stress ratio causing shear failure of the soil. The magnitude and the variation of the theoretical stress ratios with distance from the caissons are similar to those of the observed penetration resistance ratios. The theoretical size and shape of the zones of soil compaction near the caissons also compare well with those observed; the zones for the deep caissons (Fig. 6) are about twice as large as for the shallow caisson (Fig. 7), as would be expected from the ratio of their depths below the surface.

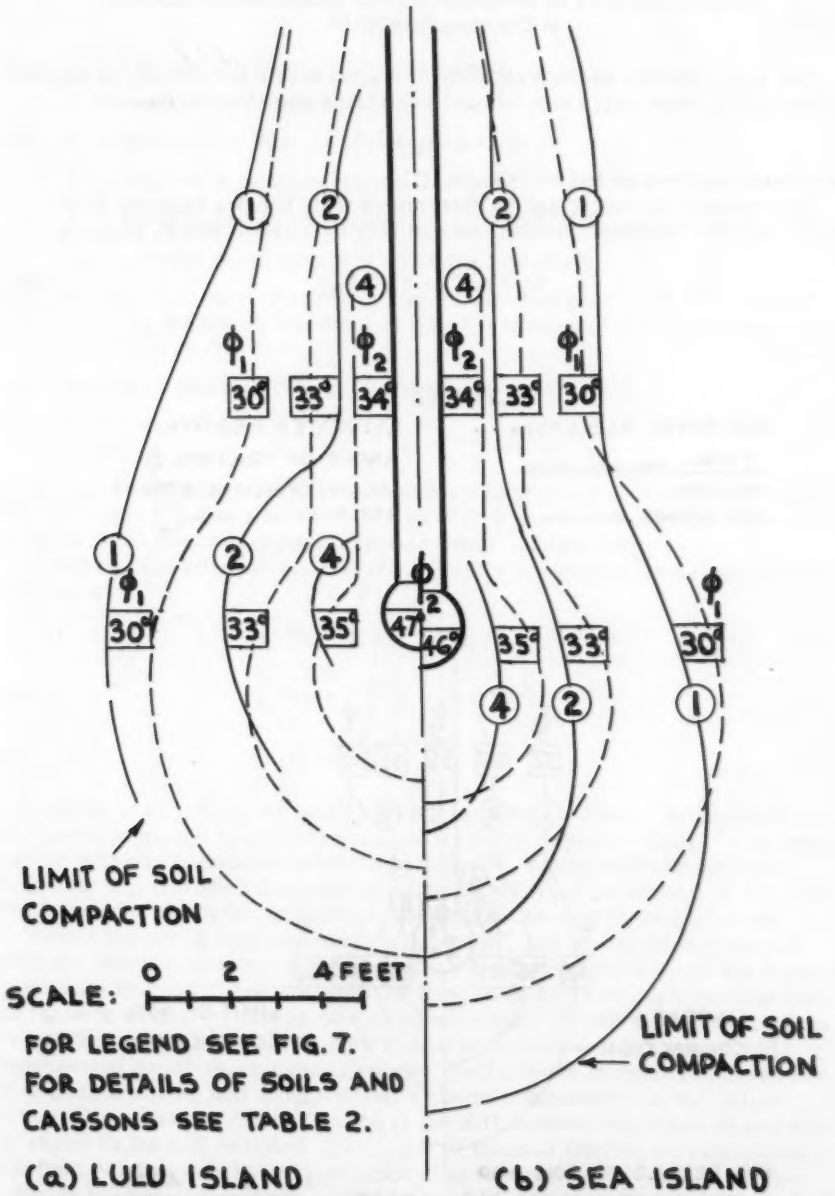

FIG. 6. COMPACTION OF SAND NEAR CAISSONS

Bearing Capacity of Driven Piles and Displacement Caissons in Cohesionless Soils

The total ultimate bearing capacity of a pier, single buried pile or caisson with a poured base is the sum of base resistance and skin friction or

$$Q_1 = q_1 A_1 + f_1 S_1 \tag{14}$$

with basic symbols as before (see Eq. (7)).

The values of q_1 and f_1 can be determined from bearing capacity theory[6,7] and for uniform cohesionless soil they are, respectively, given by

$$q_1 = \gamma \frac{BN}{2} \gamma + K_b \gamma D N_q \tag{15}$$

and

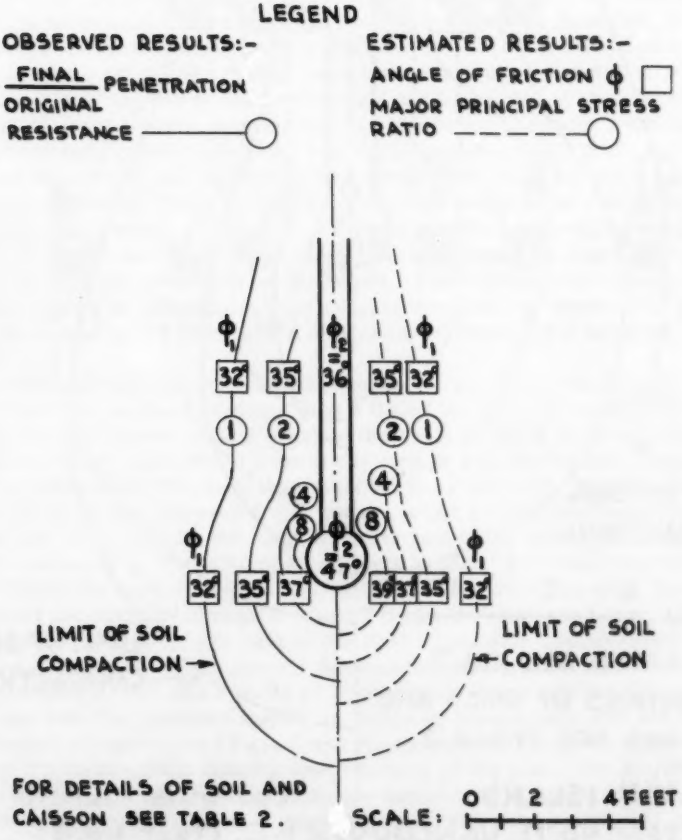

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FIG. 7. COMPACTION OF SAND NEAR CAISSON

$$f_1 = \frac{K_s}{2}\gamma D \tan\delta \qquad (16)$$

where γ = average effective unit weight of soil,

δ = angle of skin friction,

B = diameter of base,

D = depth of base below ground surface,

K_b = earth pressure coefficient on shaft within failure zone near base,

K_s = average earth pressure coefficient on shaft,

and N_γ and N_q = bearing capacity factors for deep pier ($D/B \geq 10$) depending mainly on the angle of internal friction of soil in failure zone near base.

For a foundation depth $D/B < 10$, as shown previously,(6,7)

$$q_1 = \frac{D}{10B}\left(\gamma\frac{BN}{2}\gamma + K_b\gamma D N_q\right) \qquad (17)$$

and for an embedment of D' into a cohesionless stratum underlying cohesive soil, the depth D' should be used instead of D in the factor outside the brackets of Eq. (17) but the total depth D remains within the brackets.

Similarly, the ultimate bearing capacity of a driven pile or a displacement caisson is

$$Q_2 = q_2 A_2 + f_2 S_2 \qquad (18)$$

where

$$q_2 = \gamma'\frac{BN'}{2}\gamma + K_b'\gamma' D N'_q \qquad (19)$$

and

$$f_2 = \frac{K'_s}{2}\gamma' D \tan\delta' \qquad (20)$$

with symbols as before but now referring to compacted soil. Alternatively, the values of q_2 and f_2 can be estimated from the results of static (deep sounding) or dynamic (standard) penetration tests.(9) For an embedment ratio $D/B < 10$, Eq. (19) should be multiplied by D/10B in accordance with Eq. (17), while for deep bases the width term in Eq. (19) can usually be neglected.

Before the above bearing capacity factors N'_γ and N'_q can be determined from an extended bearing capacity theory which takes into account the increase in internal friction of the soil near the base, it is necessary to estimate the increase in density by compaction of the material. If the compaction properties of the cohesionless soils underlying a particular site are known and represented by pressure-density curves, the increase in relative density due to driving a pile or installing a caisson can be estimated from the major principal stresses at various points in the soil surrounding the shaft and base, as shown in the last sections. The angle of internal friction corresponding to the final relative density is then obtained at various points and used to determine the bearing capacity factors N'_γ and N'_q which include the effect of an increase of internal friction of the compacted soil near the base. The previous bearing capacity theory of foundations for soils of constant angle of internal friction (6,7) can be extended to soils of variable angle of internal friction provided the angle ϕ changes with distance from the base, but a

solution of this problem has so far only been obtained for a linear variation of ϕ using a numerical procedure of computation.

If the friction angle ϕ is assumed to increase linearly from a lower value of ϕ_1 (corresponding to relative density D_1) at the failure surface to an upper value of ϕ_2 (corresponding to relative density D_2) at the base, the shear pattern in the plastic zones consists of curved radial and tangential slip lines (Figs. 3 and 4). The plastic equilibrium in the shear zones can be determined along every tangential slip line along which the angle of internal friction is constant in accordance with the above assumption. From the stresses the passive pressure can then be computed along various sections of the curved radial shear surface near the base where the angle of inclination with the horizontal varies from $45^\circ + \phi_2/2$ at the perimeter to $45^\circ + \phi_1/2$ at the lower point. The vertical component of the passive pressure, which was found to increase from a minimum at the perimeter of the base to a maximum near the quarter-point and then decrease again to the centre of the base, gives the required ultimate bearing capacity of the base.

From detailed computations of the bearing capacity factors N'_γ and N'_q for various conditions it has been found that an equivalent constant angle of internal friction ϕ' of the compacted soil within the failure zone can be determined, which can be used with the previous factors N_γ and N_q to give a close and safe estimate of N'_γ and N'_q, respectively. This angle ϕ' was found to be at the centre of pressure of the ϕ variation diagram for N'_γ and at the centre of gravity of this diagram for N'_q. Thus, for N'_γ

$$\phi' = \phi_2 - \frac{\text{second moment about base of } \phi \text{ diagram}}{\text{first moment about base of } \phi \text{ diagram}} \times \frac{\phi_2 - \phi_1}{\text{height of } \phi \text{ diagram}} \qquad (21)$$

and for N'_q

$$\phi' = \phi_2 - \frac{\text{first moment about base of } \phi \text{ diagram}}{\text{area of } \phi \text{ diagram}} \times \frac{\phi_2 - \phi_1}{\text{height of } \phi \text{ diagram}} \qquad (22)$$

For a linear variation of ϕ as used above

$$\phi' = \frac{(\phi_1 + \phi_2)^2/2 + \phi_1^2}{2\phi_1 + \phi_2} \quad \text{for } N'_\gamma \qquad (23)$$

$$= \frac{2\phi_1 + \phi_2}{3}, \text{ approximately} \qquad (23a)$$

and

$$\phi' = \frac{(\phi_1 + \phi_2)^2 - \phi_1\phi_2}{1.5(\phi_1 + \phi_2)} \quad \text{for } N'_q \qquad (24)$$

$$= \frac{\phi_1 + \phi_2}{2}, \text{ approximately} \qquad (24a)$$

To simplify computations in practice the bearing capacity factors N'_γ and N'_q have been computed for various angles of internal friction from 30° to 45° using the method given above and assuming a linear variation of ϕ from a lower value of ϕ_1 at great distance from the base to an upper value of ϕ_2 at the base. The final bearing capacity factors are given in Fig. 8 and indicate

the considerable increase in theoretical bearing capacity produced by the greater angle of internal friction of the compacted soil.

In addition, the ultimate bearing capacity depends on the unit weight of the compacted soil and the earth pressure coefficient on the shaft, which includes the effect of prestressing of the soil mass near the base and shaft. The average effective unit weight γ' of the compacted soil may be taken as that corresponding to the average relative density $(D_1 + D_2)/2$. While the average earth pressure coefficient K'_s on the shaft of driven piles was found[6] to vary from about 0.5 for loose sand (ϕ_1 about 30°) to 1.0 for very dense sand (ϕ_1 about 45°), analysis of model tests[15] and field observations[16] of driven piles in sands shows that the earth pressure coefficient K'_b on the shaft within the failure zone near the base is about 70 per cent of the average value K'_s. Thus,

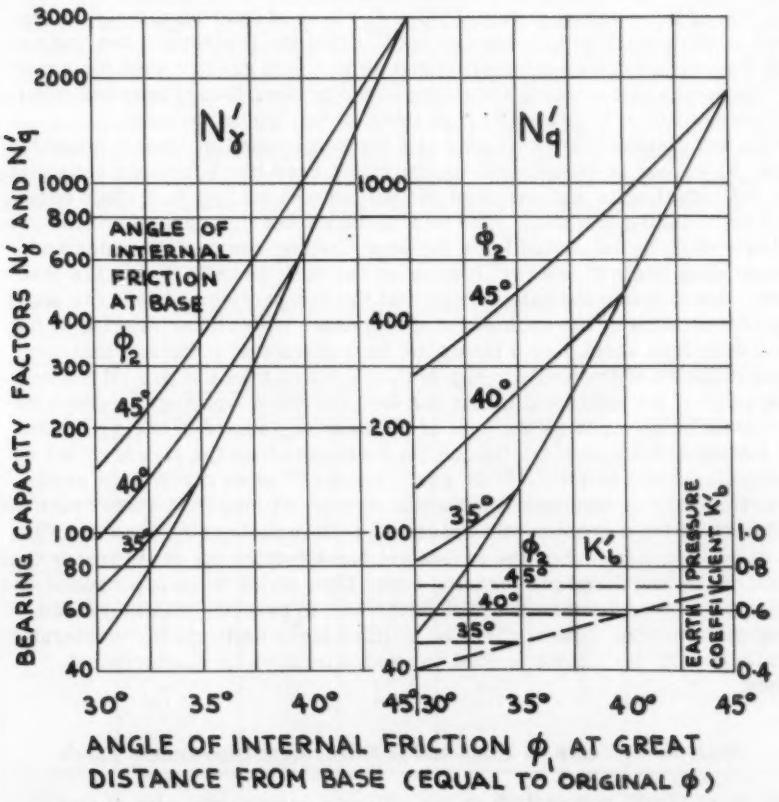

FIG. 8. BEARING CAPACITY FACTORS FOR DRIVEN PILES AND DISPLACEMENT CAISSONS IN COHESIONLESS SOILS

the earth pressure coefficient K'_b may be taken to vary from about 0.4 for $\phi_1 = 30^\circ$ to 0.7 for $\phi_1 = 45^\circ$, as shown in the lower part of Fig. 8.

Thus, for a driven pile in loose sand (ϕ_1 about 30°) with a 5° increase of internal friction at the base (ϕ_2 about 35°) the theoretical point resistance (and skin friction) would be about 60 per cent greater than for a buried pile; since a further increase in bearing capacity due to prestressing is about 25 per cent, the gross estimated bearing capacity is then about double that without soil compaction (Fig. 8). A similar increase in bearing capacity would be expected theoretically for a pile driven into compact sand (ϕ_1 about 35°). On the other hand, for a displacement caisson in loose sand with a 15° increase of internal friction at the base, the theoretical base resistance is about 4 times that of a poured base or, including the 50 per cent increase due to prestressing of the soil, the gross bearing capacity is estimated to be about 6 times that of the original material (Fig. 8). It is of interest to note that an earlier semi-empirical approach showed[7] that the theoretical bearing capacity factors for driven piles are about 60 per cent greater than for corresponding buried piles, which according to the present theory corresponds to an increase of 4° to 5° in the angle of internal friction at the base.

Piles and displacement caissons are frequently used in closely spaced groups, which can be considered as equivalent deep block footings consisting of the individual units and enclosed compacted soil resting in further compacted surrounding material. The total ultimate bearing capacity of the group could therefore be estimated from the over-lapping compaction zones and corresponding angle of internal friction of the soil, as indicated in the last section. Since it was indicated above that the compaction and thus the angle of internal friction of the enclosed soil increases with closer spacing of the units of less than about 7 or 8 times the base diameter, it follows that the bearing capacity of the whole group is then greater than the sum of the bearing capacity of the individual units; and for very close spacing the group capacity could be as much as the sum of the base resistance of the equivalent block footing and the skin friction on its perimeter less the weight of the enclosed soil. Model tests[17,18] on small groups of piles driven into sands have confirmed this conclusion for a pile spacing of less than about twice the pile diameter; for a greater pile spacing the ultimate bearing capacity was found to be less than that of the equivalent block footing but still greater than the sum of the individual pile bearing capacities, which were approached at a pile spacing of 6 or 7 times the pile diameter. In practice this increased bearing capacity can, however, not be utilized since settlement considerations generally govern the allowable load on groups of piles or displacement caissons.

Analysis of Tests on Piles and Displacement Caissons in Sands

The effect of soil compaction on the ultimate bearing capacity of single driven piles can best be ascertained from a comparison of the results of loading tests on similar driven and buried piles on the same site. In the absence of full-scale tests, the results of a series of model tests showed[6] that for loose sand the ultimate bearing capacity of driven piles is nearly twice that of buried piles. Since ancillary measurements showed that for the driven piles the shaft pressure coefficient was about 30 per cent greater than for buried piles, the angle of internal friction at the shaft of the driven piles is deduced to be 4° to 5° greater than for corresponding buried piles.

These findings are supported by the results of static cone penetration tests near driven piles on 2 sites,[10,11] when the average penetration resistance in the failure zone near the base was found to be 2 to 3 times greater than the original value (see also Fig. 5). The ultimate bearing capacity would, therefore, be expected to be similarly increased compared with the uncompacted material since the bearing capacity of piles is directly proportional to the penetration resistance in the failure zone.[9,10,13] Moreover, at the pile base the angle of internal friction deduced from the observed cone resistance is about 4° to 6° greater than the original friction angle, while the friction angle at the shaft is about 4° greater than the original value (Fig. 5).

A further check of the proposed method is possible from an analysis of the results of loading tests of driven piles on several sites where the shearing strength of sand is known.[16,19,20,21,22] The soil and foundation conditions of these sites are given in Table 1 together with the corresponding estimates of the friction angles deduced from the loading tests using Eqs. (18) to (20) with a 4° increase of the angle of internal friction at the piles and using the earth pressure coefficients given in the last section except for the Old River site when the deduced values of this coefficient have been used. It is found that the estimated angles of internal friction at great distance from the piles are in fair agreement although frequently somewhat smaller than those obtained from laboratory shear tests on the same or similar material.

TABLE 1

RESULTS OF LOADING TESTS ON DRIVEN PILES IN SANDS

Description of Soil	Pile: Type and Diameter B	Pile: Embedded Length D	Pile: Ultimate Load Q	Angle of Internal Friction: Actual ϕ	Angle of Internal Friction: Estimated ϕ_1	Location and Reference
	inches	feet	tons	degrees	degrees	
Loose Fine Sand	Timber 8 (tip)	27	59	35	35	Aalborg, Denmark Mortensen (1948)
Loose Silty Sand	Concrete 12 sq.	43	72	34	32	Larvik, Norway Eide (1956)
Compact Fine Sand	Steel (1) 24	9	240	c.35	33	Morganza, La. Mansur and Focht (1953)
	(2) 8(tip)	7.6	90	c.35	33	
	(3) 12	1.9	88	c.35	35	
	(4) 18	3.5	180	c.35	34	
	(5) 24	5.7	205	c.35	33	
	(6) 30	6.1	286	c.35	34	
	Concrete (7) 22 sq.	8.8	170	c.35	32	
Compact Medium Sand	Steel (1) 14 (H)	33	185	36	34	Old River, La. Mansur and Kaufman (1956)
	(2) 21	17	165	36	32	
	(3) 14 (H)	19	135	36	36	
	(4) 17	18	250	36	32	
	(6) 19	15	135	36	33	
Dense Fine Sand	Timber 14 sq.	2.3	175	45	45	London, England Meyerhof (1953)
	Concrete 10 sq.	5	140	42	40	

For the purpose of comparing the proposed theory with the results of field observations on single displacement caissons, full details of the results of soil explorations including penetration tests and laboratory investigations have been obtained for a number of sites from the Franki of Canada Limited for analysis together with the results of driving the casing tubes, expanding the bases and loading tests on the caissons. To facilitate a more detailed analysis of the soil compaction penetration tests were made in a few cases at various distances from the caissons, as mentioned earlier. The main soil and foundation conditions at the various sites used in the analyses are given in Table 2. For the analysis it is convenient to consider each of the three main stages in the construction of typical displacement caissons, namely the first stage when the plugged casing tube has been installed to its design depth, the second stage when the base has been expanded to its full size and the third stage when the completed caisson has been loaded.

To analyse the first stage the angle of internal friction ϕ_1 of the soil at great distance from the casing is deduced from the final driving resistance of the plugged casing using Eqs. (9) and (18) to (20) with a 4° increase of the angle of internal friction at the casing and the earth pressure coefficients suggested above. This friction angle ϕ_1 is then compared with the value of ϕ obtained from laboratory tests or deduced from the standard penetration resistance.(9) Similarly, in analysing the second stage the friction angle ϕ_2 at the base is deduced from the energy per blow required for final expansion of the base using the same equations with the above value of ϕ_1 at great distance from the base and assuming a linear variation of ϕ in the failure zone. This friction angle ϕ_2 is then compared with that deduced on the basis of Eqs. (18) to (20) from the ultimate loads, which in all cases had to be extrapolated from the loading tests since they were carried out only to about twice the design load. In addition the deduced friction angle ϕ_2 can be compared with the maximum value of $\phi_2 = 45°$ for very dense sand at the base; moreover the variation of ϕ near the shaft and base can be estimated from the penetration tests made at various distances from the caissons on 3 of the above sites, as mentioned earlier, and the results may be compared with the linear variation of ϕ assumed theoretically.

The results of these various analyses are summarized in Table 2, which shows that the theoretical angle of internal friction ϕ_1 deduced from the final driving resistance of the casing agrees generally within about 1° with the angle ϕ indicated by the standard penetration resistance or by the laboratory tests. Since the deduced effective angle of internal friction ϕ' near the casing is found to be on the average 2° to 3° greater than the friction angle ϕ indicated by laboratory shear or standard penetration tests, it may be concluded that installing the casing increases the angle of internal friction ϕ_2 on the average by about 4° to 6° at the casing. The theoretical angle of internal friction ϕ_2 at the base deduced from the final resistance to expansion of the caisson base is found to vary from about 38° (dense sand) to 47° (very dense sand) (Table 2). The results also show that for a constant number of blows per cubic foot of base concrete the angle ϕ_2 increases with the energy per blow, as would be expected theoretically, and that the energy per blow should not be less than about 70 ft.-t. with 4 blows/cu. ft. of concrete in order to obtain a very dense material ($\phi_2 = 45°$) near the base at a depth of about 30 ft. Since the effective angle ϕ' near the base increases with the energy per blow used for expanding the base at a given number of blows, the bearing capacity of the caisson should similarly increase. While the observations give some support to these

relationships, the angle of internal friction ϕ_2 at the base deduced from the extrapolated ultimate bearing capacity of the caisson is about 5° smaller than the corresponding value indicated by the expansion of the base, and in some cases this angle is only equal to the value indicated by the installation of the corresponding casing, which is impossible (Table 2). It may, therefore, be concluded that the extrapolation of the loading tests is conservative and loading tests to failure are required before a complete comparison is possible between theory and observation.

The results of the penetration tests near the caissons on 3 sites mentioned above (Fig. 6 and 7) show that the average penetration resistance in the failure zone near the base is about 6 times greater than the original value so that a similar increase in the ultimate bearing capacity would be expected compared with uncompacted material. The angle of internal friction ϕ estimated from

TABLE 2

RESULTS OF INSTALLATION AND LOADING TESTS ON DISPLACEMENT CAISSONS

Test No.	Location	Description of Soil (Standard Penetration Resistance) (N) (blows/ft.)	Installation Details of Caisson				
			Casing (Shaft)			Base	
			Diameter B inches	Length D feet	Energy/Blow (Blows/Foot) Wh (n) ft-tons (no.)	Diameter B inches	Energy/Blow (Blows/Cu.Ft.) Wh (n) ft-tons (no.)
1	Brooklyn, N.Y.	Loose Coarse to Fine Sand (N = 7)	20	20	52 (19)	48	32 (12)
2	Brooklyn, N.Y.	Loose Coarse to Fine Sand (N = 9)	20	40	35 (21)	48	32 (15)
3	Fair Lawn, N.J.	Compact Fine Sand (N=14)	20	18.5	70 (10)	36	70 (4)
4	Rio De Janeiro, Brazil	Loose Fine to Medium Sand (N = 9)	16	26.5	47 (6)	44	45 (6)
5	Rio De Janeiro, Brazil	Loose Fine to Medium Sand (N = 8)	16	43.5	47 (10)	40	45 (6)
6	Sorel, Que.	Loose Fine Silty Sand (N=5)	20	26	56 (8)	48	56 (4)
7	Vancouver, Lulu Isl.	Loose Medium Sand (N=6)	20	31.5		32	70 (6)
8	Vancouver, Sea Isl.	Loose Medium Sand	20	32	84 (5)	36	70 (5)
9	Worcester, Mass.	Compact Coarse Sand (N=13)	20	21	70 (10)	42	70 (4)
10	Worcester, Mass.	Compact Coarse Sand (N=14)	20	15	70 (16)	36	70 (5)
11	Worcester, Mass.	Loose Coarse Sand (N=7)	16	17	50 (4)	32	50 (4)

the observed penetration resistance at various distances from the caissons is also given in Figs. 6 and 7, and the limiting values are found to be in reasonable agreement with those deduced at great distance (ϕ_1) and near the caisson shaft and base (ϕ_2) from the analyses of the energy per blow required for installing the casing and expanding the base (Table 2). The friction angles increase roughly linearly towards the shaft and towards the base at some distance from the base. Within a zone of about one base diameter from the base, however, the angles of internal friction increase rather more rapidly towards the base and an approximately parabolic variation is found. In the latter zone the major principal stresses causing compaction of the soil increase, therefore, rather more rapidly than had been assumed theoretically.

TABLE 2 (Continued)

RESULTS OF INSTALLATION AND LOADING TESTS ON DISPLACEMENT CAISSONS

Test No.	Load Test	Angle of Internal Friction				
	Extrapolated Ultimate Load	Actual	Estimated			
			Original	Final		
				Shaft	Base (Expansion)	Base (Load Test)
	Q	ϕ	ϕ_1	ϕ_2	ϕ_2	ϕ_2
	tons	degrees	degrees	degrees	degrees	degrees
1	1000	c.30	35	39	41	39
2	1600	c.30	31	35	38	40
3	950	35	34	38	46	42
4	500	c.30	32	36	41	37
5	650	c.30	30	34	41	36
6	450	c.30	31	35	42	35
7	700	c.30	c.30	c.34	c.47	c.42
8	700	30	30	34	46	42
9	650	c.35	34	38	45	38
10	800	c.35	36	40	47	42
11	500	c.30	32	36	47	42

Settlement of Piled Foundations in Sands

The allowable load on a single pile and displacement caisson or a group of units must not only provide an adequate margin of safety on the ultimate bearing capacity of the base or shaft, as indicated in the previous section. The allowable load must also limit the differential settlement to a tolerable amount, and it has been suggested[5] that for ordinary structures a maximum settlement of 1 in. can be allowed for footings and 2 in. for rafts in order to keep differential movements within 3/4 in. The degree of compaction of the soil at the base of a driven pile or a displacement caisson is much higher than at greater distance from the base. The allowable load on a single base will therefore be given either by a high allowable base pressure (corresponding to the average density of the compacted soil near the base) multiplied by the base area, or it will be given by a low allowable bearing pressure (corresponding to the original density of the soil) multiplied by the corresponding bearing area which in accordance with the extent of the theoretical failure zone has a minimum width of 4 times the base diameter. Trial computations have shown that the former condition governs the allowable load; they have also indicated that the seat of settlement is within a depth of twice the base diameter below the base in the present case where the compressibility of the compacted sand increases with distance from the base.

Using the suggested relationships between the standard penetration resistance and the allowable bearing pressure of foundations of various sizes and depths in sand,[5,9] the allowable base pressure has been computed for a settlement of 1 in. of a single base in uniform cohesionless soils of various relative densities and not underlain by more compressible material at greater depths. The results of these computations show that for shallow bases of up to about 20 ft. depth bearing capacity frequently controls the allowable load, while for deep bases settlement usually governs. Since the modulus of deformation of sands was found[23] to be roughly proportional to the cone penetration resistance within the seat of settlement and since this penetration resistance also governs largely the ultimate bearing capacity of the base, it follows that compaction of the material near the base should theoretically increase the modulus of deformation by about 2 times for driven piles and about 6 times for displacement caissons compared with that of uncompacted soil, as indicated before. The observed increase of penetration resistance near driven piles (Fig. 5) and displacement caissons (Figs. 6 and 7) supports this conclusion; within the seat of settlement of single driven piles soil compaction has increased the penetration resistance by about 2 to 3 times, while for single displacement caissons the corresponding increase is found to be 5 to 6 times on account of the greater degree of compaction of the soil. In both cases the upper limit applies to the originally looser sand and the lower limit to originally more compact material.

The settlement of a single unit under load is best determined from a representative loading test on the particular site. However, a rough guide can be obtained from an analysis of the loading tests on single driven piles and displacement caissons mentioned in the last section and previous data on driven piles in sands.[24] This analysis shows that for loads less than about one-third of the ultimate the settlement of a single pile or caisson in cohesionless soil not underlain by softer soils is, approximately, given by

$$S_1 = \frac{pB}{30q} \qquad (25)$$

or $$= \frac{B}{30F} \qquad (25a)$$

where p = base pressure,

q = ultimate bearing capacity (or static cone resistance near base),

B = diameter of base

and F = factor of safety (≥ 3) on ultimate load.

The allowable load on groups of driven piles and displacement caissons can be estimated as for a single unit with allowance for overlapping compaction zones, as indicated above. Trial computations have shown that for large groups of units spaced less than about 7 or 8 times the base diameter the allowable load in otherwise homogeneous cohesionless soils of great depth is given by the allowable bearing pressure (corresponding to the original density of the soil) multiplied by the corresponding bearing area of the whole group. Using the above mentioned relationships between standard penetration resistance and allowable bearing pressure of deep foundations of various widths in sand, the computations have shown that for large groups of driven piles of about 1 ft. diameter or of displacement caissons of about 3 ft. diameter the allowable load per unit is only about 5 per cent of that of a single unit if the total settlement of the group is to be limited to 1 in. or 10 per cent of that of a single unit of 2 in. settlement can be tolerated. The results of the computations also show that the allowable load per pile or caisson in a group increases rapidly with smaller spacing of the units and as the size of the group decreases, because the seat of settlement lies then in more compacted soil. For a uniform spacing of driven piles or displacement caissons of 2 to 6 times their base diameter, the allowable load per unit in a square group is found to be approximately given by

$$Q_g = \frac{(1 + 1/r)^2}{s(5 - s/3)} \times Q_1 \qquad (26)$$

where r = number of rows of units,

s = ratio of spacing to diameter of units

and Q_1 = allowable load on single unit.

Conversely, the computations indicate that the settlement of a group increases with the width of the group and spacing of the units, and the settlement of a large group with widely spaced units may be up to about 20 times that of a single pile or caisson under the same load per unit, as shown by the curves in Fig. 9. Thus, it follows from Eq. (26) for the same limits of spacing of the units in cohesionless soils that the settlement of a square group is approximately given by

$$S_g = \frac{s(5 - s/3)}{(1 + 1/r)^2} \times S_1 \qquad (27)$$

or, in terms of the foundation width L,

$$S_g = \frac{(1 + L/sB)^2\, s(5 - s/3)}{(2 + L/sB)^2} \times S_1 \qquad (28)$$

where S_1 = settlement of single unit (under same load as unit in group) and other symbols as before.

Substituting Eq. (25a) into (27), the settlement of a group is approximately

$$S_g = \frac{s(5 - s/3)}{(1 + 1/r)^2} \times \frac{B}{30F} \qquad (29)$$

Thus, for piled footings and a maximum settlement of 1 in., the allowable load per unit requires a factor of safety of about

$$F = \frac{s(5 - s/3)B}{30(1 + 1/r)^2} \qquad (30)$$

where B is in inches, while for a piled raft and a maximum settlement of 2 in. the required factor of safety would be one-half of that given by Eq. (30). According to this relationship the required factor of safety on the ultimate load of a single unit varies from about 3 (minimum) for small groups and closely spaced units of small diameter (driven piles) to about 10 for large groups and widely spaced units of large diameter (displacement caissons); these values are of the same order as commonly used in practice and give the allowable loads customary for such units (Fig. 9).

As a check of these estimates the results of published field observations on groups of driven piles and Franki displacement caissons in sands[20,22,25,26,27,28] together with some unpublished observations have been analysed and the results are shown in Fig. 9. It is found that the ratio of the observed settlement of the whole group to the settlement of a single pile or caisson under the same load in a loading test increases rapidly but at a decreasing rate with the number of rows of units in the group, i.e. with the ratio of the width of the group to the spacing of the units. At a given number of rows of units the settlement ratio increases also with the ratio of spacing to base diameter of the units; for very large groups of units spaced at 4 times the base diameter the maximum settlement ratio is found to be 17. For the maximum theoretical spacing ratio of 7 to 8, however, the settlement ratio might be expected to approach about 20, as had been estimated from the compaction of the

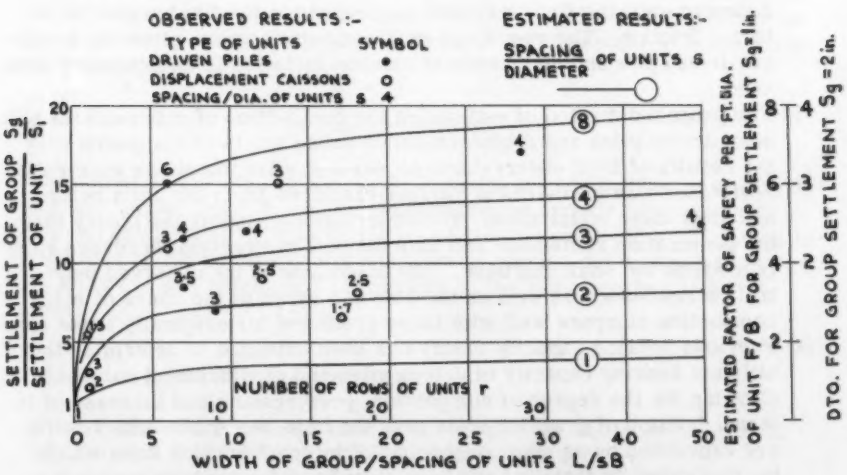
OBSERVED RESULTS :-
TYPE OF UNITS SYMBOL
DRIVEN PILES •
DISPLACEMENT CAISSONS o
SPACING/DIA. OF UNITS s 4
ESTIMATED RESULTS:-
SPACING / DIAMETER OF UNITS s
SETTLEMENT OF GROUP Sg / SETTLEMENT OF UNIT S1
NUMBER OF ROWS OF UNITS r
WIDTH OF GROUP/SPACING OF UNITS L/sB
ESTIMATED FACTOR OF SAFETY PER FT. DIA. OF UNIT F/B FOR GROUP SETTLEMENT Sg = 1 in.
DTO. FOR GROUP SETTLEMENT Sg = 2 in.

FIG. 9. SETTLEMENT OF PILED FOUNDATIONS IN SANDS

material near the base. The scatter of some of the observed settlements may be explained by local variations of the soil properties near the piles used for loading tests and by some differences in the shape of the foundation in plan because rectangular foundations produce larger stress zones leading to greater settlement ratios compared with square foundations of the same width.

CONCLUSIONS

(1) The bearing capacity of piled foundations in cohesionless soils is frequently estimated from static formulas which ignore the influence of compaction and prestressing of the soil by pile driving. In order to provide information on the compaction properties of granular soils under impact, a laboratory study has been made on loose sand at various moisture contents. The results show that the relative density of the soil increases with the peak pressure at a decreasing rate and that impact pressures produce greater densities than corresponding static pressures. Analysis of these tests and previous field experiments with rolling and vibrating compaction equipment shows that the experimental density-pressure relationships can be expressed in terms of similar parameters to those of a void ratio-pressure curve but using a compaction index and pressure constant depending on the physical properties of the soil and the method of compaction.

(2) A method has been developed to estimate the degree of compaction of sands near a driven pile or base of a displacement caisson under the pressures induced by the energy of the hammer. From empirical impact pressure-density relationships of the soil the increase in relative density is estimated from the major principal stresses computed at various points in the soil surrounding the pile or caisson. The angles of internal friction corresponding to the new relative density are then obtained and used to recalculate the principal stresses; this process is repeated until the final stresses correspond to the final angles of internal friction. The results of such computations are given for a typical driven pile and displacement caisson installed in an originally loose sand.

(3) The proposed method of estimating the compaction of cohesionless soils near driven piles and displacement caissons has been compared with the results of field observations on several sites where the penetration resistance was measured at various distances from the units before and after their installation. The observations support the theory that the penetration resistance and thus the soil compaction increases rapidly towards the shaft and base. The magnitude of the observed penetration resistance as well as the size and shape of the zones of soil compaction compare well with those predicted for originally loose sand.

(4) Previous bearing capacity theory has been extended to determine the ultimate bearing capacity of driven piles and displacement caissons allowing for the degree of compaction, prestressing and increase of internal friction of granular soils near the base and shaft. The results are expressed by an equivalent angle of internal friction from which bearing capacity factors have been calculated for various angles of internal friction in practice on the assumption that compaction of the soil

produces a linear variation of the friction angle near the base. The theory indicates that for a pile driven into loose sand the ultimate bearing capacity may be about double that without soil compaction, while for a displacement caisson the corresponding bearing capacity would be about six times that of uncompacted material.

(5) The results of loading tests on driven piles and displacement caissons in cohesionless soils have been analysed, and the angles of internal friction deduced from the ultimate loads are found to be close to those determined from laboratory shear tests or estimated from standard penetration tests. The angles of internal friction deduced from the measured penetration resistance near some of the piles and caissons indicate that the friction angles increase roughly linearly towards the shaft and towards the base at some distance from the base. Within a zone of about one base diameter from the base, however, the friction angles increase rather more rapidly towards the base than had been assumed.

(6) A method has been suggested to estimate the settlement and allowable load of single piles and displacement caissons and of groups allowing for the compaction of cohesionless soils. The results of computations indicate that for single units the allowable load on shallow bases is generally controlled by bearing capacity, while settlement governs the allowable load on deep bases and on groups for which the bearing capacity was found to be greater than the sum of the individual units. The theoretical results also show that the allowable load per pile or caisson in a group decreases considerably with larger size and spacing of the units in the group and is a minimum for a piled raft. Conversely, it has been estimated that the settlement increases with the width of the group and spacing of the units; for a large group with widely spaced units the settlement may be up to about 20 times that of a single unit under the same load. The corresponding factors of safety on the ultimate load of units in a group are found to be of the same order as commonly used in practice. Analysis of field observations of the settlement of foundations on groups of driven piles and displacement caissons in sands are generally found to be consistent with these relationships.

ACKNOWLEDGMENTS

The author is indebted to Mr. L. A. Fraikin, M. ASCE, Managing Director, Franki of Canada Limited, for his encouragement and for permission to publish an analysis of various field data of his Company. The laboratory investigations on the compaction of sands were carried out by Professor V. E. Vaughan in the Soil Mechanics Laboratory of the Nova Scotia Technical College.

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UNDRAINED STRENGTH OF COMPACTED CLAYS AFTER SOAKING

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SYNOPSIS

The influence of soil structure and such structure-determining factors as molding water content and method of compaction on the strength of compacted clays after soaking is described and the variety of relationships which may exist between initial composition and strength after soaking is discussed and demonstrated. The usefulness of the stress-strain relationship of soil samples compacted wet of optimum by kneading compaction as an indicator of the probable significance of method of compaction, molding water content and strain at failure on the induced strength characteristics of compacted clays is described.

In an earlier paper(1) the authors discussed the factors determining the structure of compacted clays and the influence of structure on shrinkage, swelling, swell-pressures, stress-deformation characteristics, undrained strength, pore-water pressures and strength characteristics expressed in terms of effective stresses. These concepts were utilized to explain the influence of method of compaction on soil properties and the relationship between composition and soil strength, primarily with reference to strengths determined in the "as-compacted" condition. However, in practice, compacted soils are often soaked after compaction to a condition approaching saturation —for example, under pavements and in earth dams—and the relationships between initial composition and strength after soaking for such soils are an important practical consideration. In the following pages an attempt will be made

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to clarify some of the factors determining the undrained strengths of compacted clays under these conditions.

For clays which become saturated after compaction, interpretation of the relationship between composition and strength is somewhat simplified by the fact that the density and water content are related and a knowledge of one determines the corresponding value of the other. Thus the important compositional factors determining pore-water pressures and soil strength—dry density, water content and structure—are, in effect, reduced to two—the final water content and the soil structure.

On the other hand, the problem of determining these two factors becomes more complex than for soils in the "as-compacted" condition. The final water content of a compacted clay after a period of soaking depends on the initial structure (which in turn depends on the method of compaction, the compacted density and the water content at compaction and influences the swelling characteristics of the soil) and on the surcharge pressure. The stress-strain relationship of the soil in undrained tests depends on this final water content (and the corresponding dry density), on the structure after swelling or soaking has ceased which influences the pore-water pressures developed during loading, and on the change in structure as the sample is deformed during loading. Finally, the strength depends on the strain criterion selected as a basis for determining failure and on the form of the stress vs. strain relationship. In view of the number of variables involved, careful consideration is required in attempting to predict the influence of initial composition on the undrained strength of compacted clays under these conditions.

Relationship Between Initial Composition and Undrained Strength of Samples Soaked at Constant Volume

A special class of samples which become saturated after compaction are those soaked to saturation without change in volume. Soils falling in this category may be of two types—(1) non-expansive soils which are soaked under a low surcharge pressure and (2) expansive soils soaked under a surcharge pressure sufficiently high to prevent expansion. Under these conditions the problem of determining the influence of the compaction conditions on the final undrained strength is somewhat simplified. Samples compacted initially to the same density and soaked without change in volume end up with the same density and water content and, consequently, any differences in strength characteristics are due primarily to differences in pore-water pressures resulting from their different initial structures.[1] Since the initial structure depends on the water content at compaction, this latter factor will influence the strength of a compacted clay even though the water content is changed by soaking. In fact, a study of the relationship between dry density and water content at compaction and strength after soaking is necessary to determine the placement conditions which will result in maximum undrained strength of a compacted clay.[2]

The results of such a study on samples of silty clay (Liquid Limit = 37, Plastic Limit - 23, Per cent finer than 0.002 mm = 24) prepared by kneading compaction and soaked at constant volume are shown in Figs. 1, 2 and 3. The samples were mixed to different water contents and then compacted using three different compactive efforts. The water content and dry density of each sample were determined and then the samples were soaked under whatever surcharge was required to maintain constant volume. After a period of soaking, during which the samples became essentially saturated, the strength of

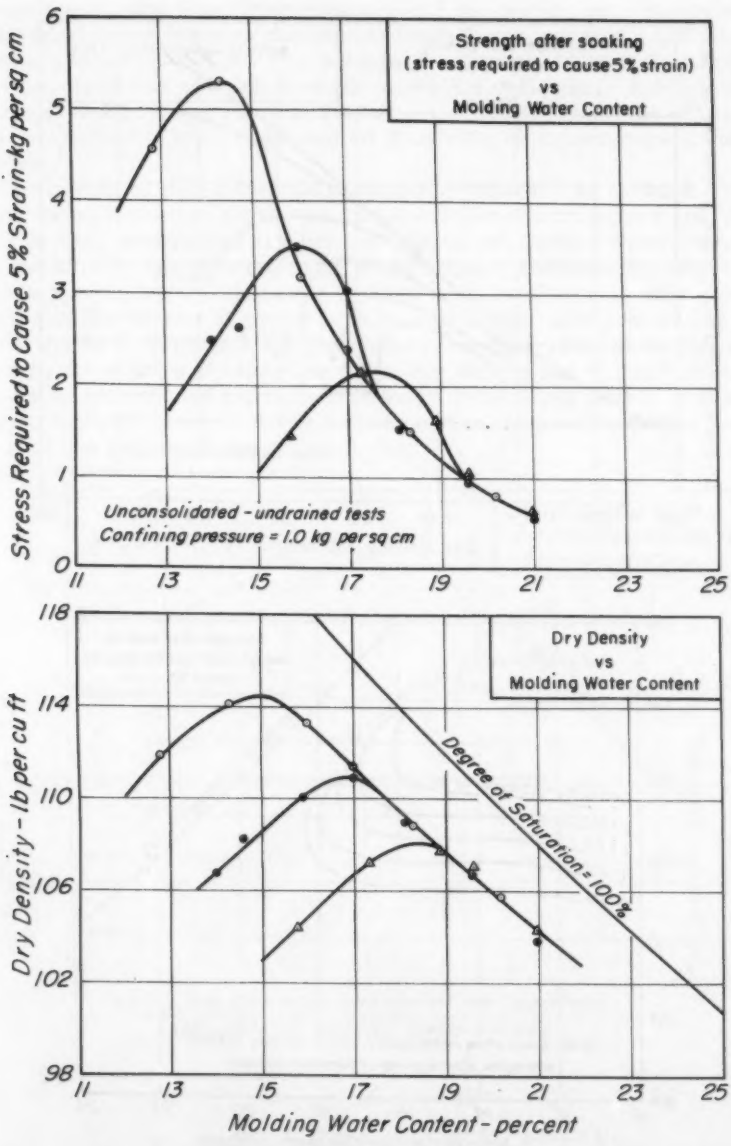
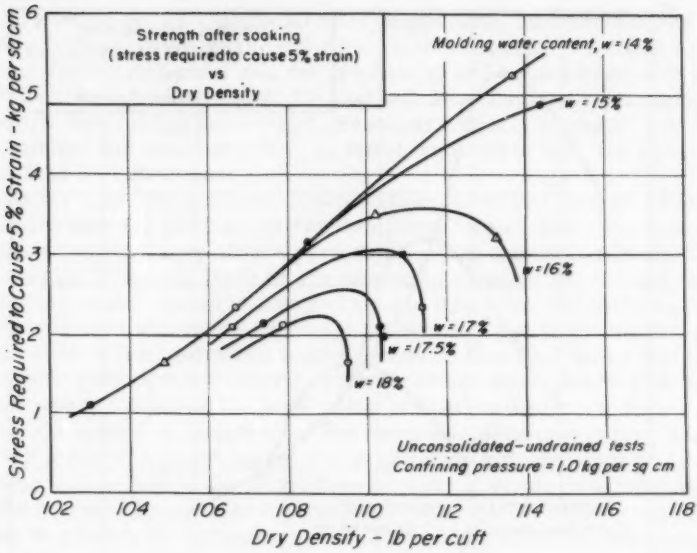

Strength after soaking
(stress required to cause 5% strain)
vs
Molding Water Content
Stress Required to Cause 5% Strain-kg per sq cm
Unconsolidated - undrained tests
Confining pressure = 1.0 kg per sq cm
Dry Density
vs
Molding Water Content
Degree of Saturation = 100%
Dry Density - lb per cu ft
Molding Water Content - percent

Fig. 1 – RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING AT CONSTANT VOLUME FOR SAMPLES OF SILTY CLAY PREPARED BY KNEADING COMPACTION.


Strength after soaking
(stress required to cause 5% strain)
vs
Dry Density
Molding water content, w = 14%
w = 15%
w = 16%
w = 17%
w = 17.5%
w = 18%
Unconsolidated-undrained tests
Confining pressure = 1.0 kg per sq cm
Stress Required to Cause 5% Strain - kg per sq cm
Dry Density - lb per cu ft

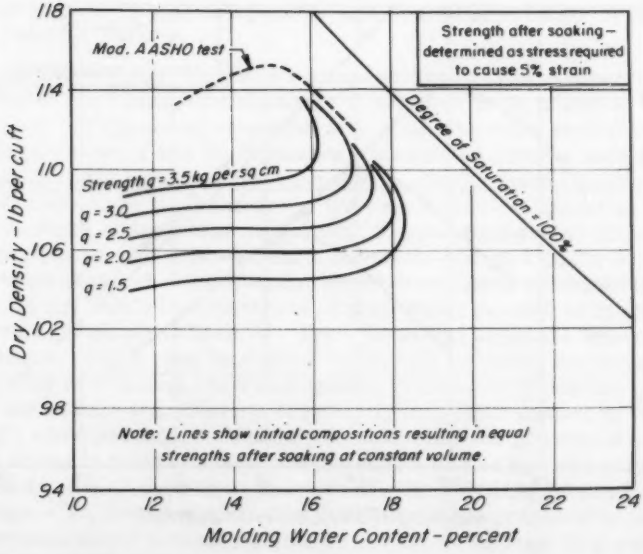

Mod. AASHO test
Strength after soaking - determined as stress required to cause 5% strain
Degree of Saturation = 100%
Strength q = 3.5 kg per sq cm
q = 3.0
q = 2.5
q = 2.0
q = 1.5
Note: Lines show initial compositions resulting in equal strengths after soaking at constant volume.
Dry Density - lb per cu ft
Molding Water Content - percent

Fig. 2 - RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING AT CONSTANT VOLUME FOR SAMPLES OF SILTY CLAY PREPARED BY KNEADING COMPACTION.

each sample was determined in an unconsolidated-undrained triaxial compression test using a confining pressure of 1 kg. per sq. cm. 'Strengths' were determined as the stresses required to cause 5 per cent strain and the stresses required to cause 20 per cent strain. The former strain corresponds approximately to that at which strength values are customarily determined for pavement design purposes while the latter is more typical of the strain used to define failure in tests conducted for foundation or embankment stability studies.

The results of such a test are customarily plotted[2] as shown in Fig. 1, which shows the initial density vs. molding water content curves and 'strength' after soaking (determined at 5 per cent strain) vs. molding water content curves for samples prepared using each compactive effort. By interpolation in this type of plot, a relationship between undrained strength after soaking and initial dry density at any given value of molding water content can readily be determined. Thus, at a selected value of molding water content, three corresponding pairs of values for initial dry density and strength after soaking can be interpolated and plotted as shown in the upper part of Fig. 2. Repeating this procedure for different values of molding water content leads to the family of curves shown in this figure.

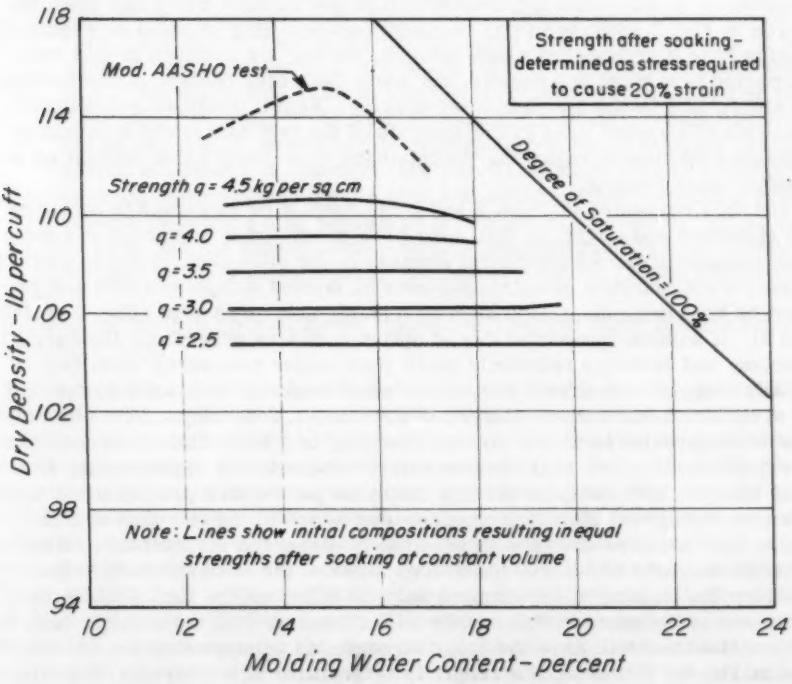


Fig. 3 - RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING AT CONSTANT VOLUME FOR SAMPLES OF SILTY CLAY PREPARED BY KNEADING COMPACTION.

Relationships of this type enable the influence of compaction conditions on final strength to be determined. However, a more convenient presentation of the same information is obtained by transferring the data to a standard plot of dry density vs. water content. Thus, by finding the points of intersection of the curves in the upper part of Fig. 2 with any horizontal line on the graph, it is possible to read off a series of corresponding values of initial dry density and molding water content which result in equal undrained strengths after soaking. These points can then be plotted as shown in the lower part of Fig. 2; repeating this procedure for different values of soil strength after soaking results in the family of curves shown in the figure.

A similar procedure, using values of soil strength after soaking determined at 20 per cent strains, led to the family of curves shown in Fig. 3.

It will be noted that the form of the resulting curves in Figs. 2 and 3 is quite different for the two definitions of soil strength. If strengths are determined at high strains, the lines are essentially horizontal, indicating that samples compacted to a given density, no matter what the water content at compaction, have the same strength. In other words, it would be concluded from this data that for this soil the water content at compaction has no influence on the soil strength after soaking.

However, if strengths are determined at low strains, say 5 per cent, the curves in Fig. 2 showing initial compositions resulting in equal strengths after soaking have a pronounced slope and indicate that for samples of this soil compacted to a given dry density, the lower the water content at compaction the higher will be the strength after soaking. Again, it will be noted that the definition of strength used in the analysis of the test data leads to markedly different conclusions regarding the influence of molding water content on soil strength after soaking.

The data presented in Figs. 2 and 3 can readily be explained in terms of soil structure and might, in fact, have been predicted in a general way from basic considerations of structural changes in the silty clay. Typical stress-strain curves obtained in undrained tests on soaked samples of this soil prepared by kneading compaction have previously been presented (Fig. 8 of Reference 1). A sample compacted dry of optimum has an essentially flocculated structure and develops relatively small pore-water pressures, resulting in an initially steep stress-strain curve even after soaking. A sample compacted wet of optimum has a more dispersed structure and develops much higher pore-water pressures at low strains resulting in a much flatter stress-strain curve. However, after soaking at constant volume to the same density and water content, both samples develop the same pore-water pressure and have the same strength at high strains, presumably due to the fact that at high strains both samples end up with essentially dispersed structures. This means that when strength is determined at high strains, the initial structure (and therefore the molding water content) have no effect on the final result; thus, specimens of the same initial density will all end up with the same density and water content and will have the same strength, as witnessed by the horizontal lines in Fig. 3. However, if strength is determined at low strains, the strength is greatly influenced by soil structure and the associated pore-water pressures and therefore by the molding water content. Further, for samples of the same final density and water content after soaking the more flocculated the structure the steeper will be the stress-strain curve and the higher will be the strength. Since, for samples compacted to the same density, the lower the water content at compaction the more flocculated the structure, it follows that the lower the

water content at compaction, the higher will be the strength at low strains (see Fig. 2).

There are two conditions to which the above reasoning would not apply:

1. If a soil is compacted in such a way that all samples have essentially the same structure, e.g. by static compaction, then all samples having the same initial density will end up after soaking at constant volume with about the same dry density, water content and structure. They will, therefore, have the same strength characteristics and consequently the lines showing initial compositions producing the same final strength will be essentially horizontal regardless of the manner in which strength is determined. A typical example of this type of relationship is shown in Fig. 4 for samples of the silty clay soil prepared by static compaction. It will be seen that, in contrast to the results for the same soil prepared by kneading compaction (Fig. 2), the molding water content has no appreciable influence on 'strengths' determined at 5 per cent strain.
2. As discussed previously(1) there are some soils which apparently retain essentially flocculated structures even when compacted wet of optimum by a method inducing large shear strains and others for which the influence of the dispersed structure on the stress-strain relationship is masked by other factors so that the soils behave as if they were flocculated. For these types of soil the strength characteristics will be similar to those discussed in 1. above. A typical example of these types of soil is the sandy clay (Liquid Limit = 35, Plastic Limit = 19, Per cent finer than 0.002 mm = 24) from Pittsburg, California, which has almost the same form of stress-strain relationship whether samples are compacted wet or dry of optimum (see Fig. 11 of Reference 1). The strength characteristics of samples of this soil soaked at approximately constant volume by using a high surcharge pressure, are shown in Fig. 5. It will be seen that the molding water content appears to have little effect on the soil strength after soaking, as indicated by the above discussion.

Influence of Initial Composition on Undrained Strength After Soaking for Highly Expansive Soils

For highly expansive soils allowed to soak under low surcharge pressures, the final water content will depend on the initial structure, water content and density. It has been shown(1) that flocculent structures tend to swell to higher water contents than do dispersed structures, and that, for soils compacted by impact and kneading methods, samples compacted dry of optimum have the more flocculated structures and end up with the highest water contents, while samples compacted wet of optimum have the more dispersed structures and, after swelling, have the lower water contents. The final undrained strength after swelling will be determined by the pore-water pressures induced during loading and these will depend primarily on the structure and final water content. Thus, for samples of the same initial density, the final strength will depend on which factor—greater degree of dispersion or lower water content—has the overriding influence on pore-water pressures and soil strength.

If strength is determined at high strains, then it seems likely that the final strength will be essentially independent of the initial structure since this will be changed to a more dispersed structure by the shear during loading and thus the strength will be governed primarily by the final water content. Since samples with the initially more dispersed structures will have the lowest final

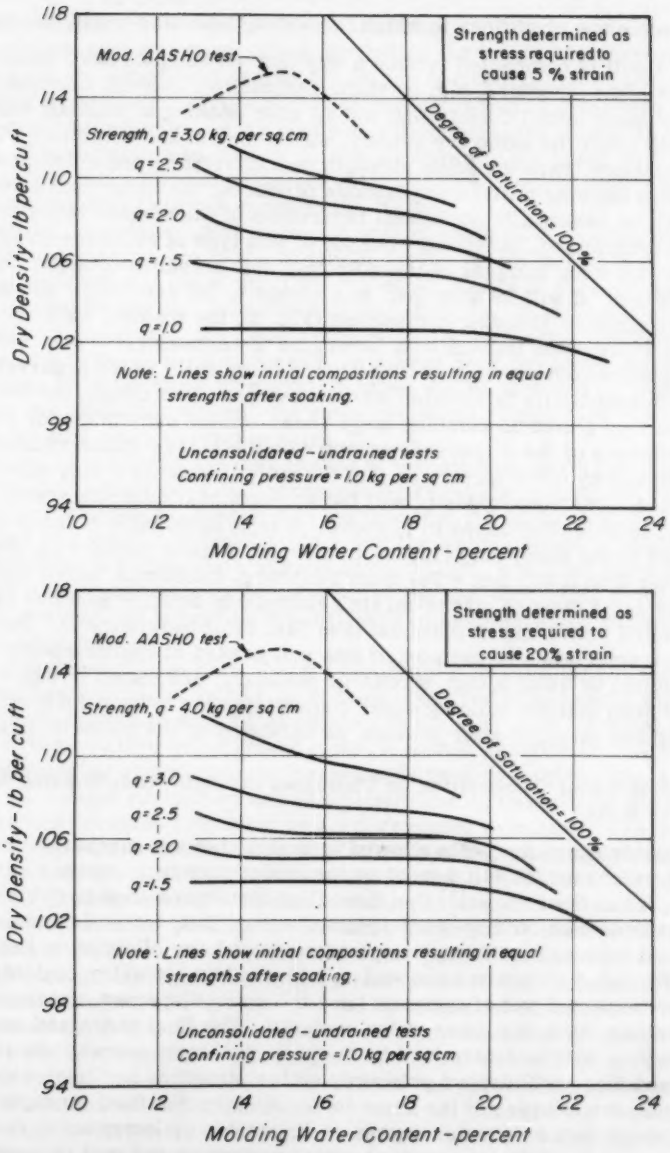

Strength determined as stress required to cause 5 % strain
Mod. AASHO test
Strength, q = 3.0 kg per sq cm
q = 2.5
q = 2.0
q = 1.5
q = 1.0
Degree of Saturation = 100 %
Note: Lines show initial compositions resulting in equal strengths after soaking.
Unconsolidated - undrained tests
Confining pressure = 1.0 kg per sq cm
Dry Density - lb per cu ft
Molding Water Content - percent
Strength determined as stress required to cause 20% strain
Mod. AASHO test
Strength, q = 4.0 kg per sq cm
q = 3.0
q = 2.5
q = 2.0
q = 1.5
Degree of Saturation = 100 %
Note: Lines show initial compositions resulting in equal strengths after soaking.
Unconsolidated - undrained tests
Confining pressure = 1.0 kg per sq cm
Dry Density - lb per cu ft
Molding Water Content - percent

Fig. 4 - RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING UNDER SURCHARGE PRESSURE OF 15 PSI FOR SAMPLES OF SILTY CLAY PREPARED BY STATIC COMPACTION.

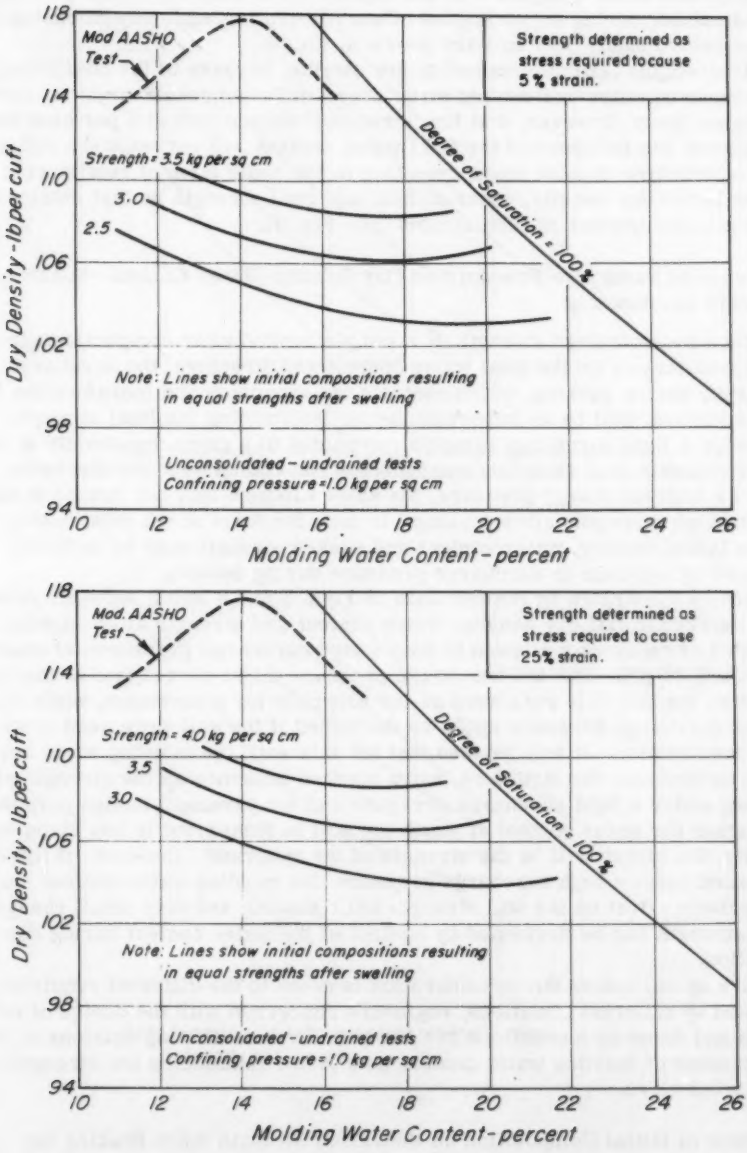


Fig. 5 – RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SOAKING UNDER SURCHARGE PRESSURE OF 35 PSI FOR SAMPLES OF SANDY CLAY PREPARED BY KNEADING COMPACTION.

water contents, it would be anticipated, for samples of the same initial dry density, that those samples with the highest water contents at compaction would exhibit the higher strengths. Data illustrating such a relationship for an expansive sandy clay soil are shown in Fig. 6.

If 'strengths' are determined at low strains, because of the conflicting influences of water content and structure, a different result might be obtained. It seems likely, however, that for 'strengths' determined at 5 per cent strain and above, the influence of the final water content will outweigh the influence of the structure in most soils, resulting in the same form of relationship between initial dry density, water content and final strength as that obtained for strengths determined at high strains (see Fig. 6).

Influence of Surcharge Pressure on Dry Density-Water Content—Soaked Strength Relationship

Since the undrained strength of a sample soaked after compaction appears to depend largely on the final water content and structure, the surcharge pressure during soaking, which controls the expansion and therefore the final water content, will be an important factor determining the final strength.

Under a light surcharge samples compacted to a given dry density at low water contents may show large expansions and end up with low strengths. Under a high surcharge pressure, the same samples may not expand at all and exhibit high strengths after soaking. In fact, the form of the relationship between initial density, water content and soaked strength may be radically changed by a change in surcharge pressure during soaking.

This is illustrated by the test data in Figs. 5 and 6 which show the relationship between initial dry density, water content and strength after soaking for samples of sandy clay allowed to soak under surcharge pressures of about 1 psi. and 35 psi. The low surcharge pressure might correspond to that imposed on the soil if it were used as the subgrade for a pavement, while the higher surcharge pressure might be developed if the soil were used in earth dam construction. It will be seen that for this soil, the molding water content, which determines the structure, has a marked influence on the strength after soaking under a light surcharge pressure and for pavement design purposes the higher the water content at which the soil is compacted to any given dry density, the higher will be the strength of the subgrade. However, if the soil is soaked under a high surcharge pressure, the molding water content has no appreciable effect on the soil strength after soaking and only small changes in final strength can be developed by control of the water content during construction.

Here again, unless due consideration is given to the different requirements imposed by different situations, engineers concerned with the design of pavements and dams or foundations are likely to form conflicting opinions of the significance of molding water content as a factor influencing the strength of compacted clays.

Influence of Initial Composition on Undrained Strength After Soaking for Moderately Expansive Soils

It will be seen from the preceding discussion that, in general, the relationship between undrained soil strength after soaking and initial composition will depend primarily on the degree of expansion which is permitted to occur (and which determines the final water content and pore-water pressures), on the

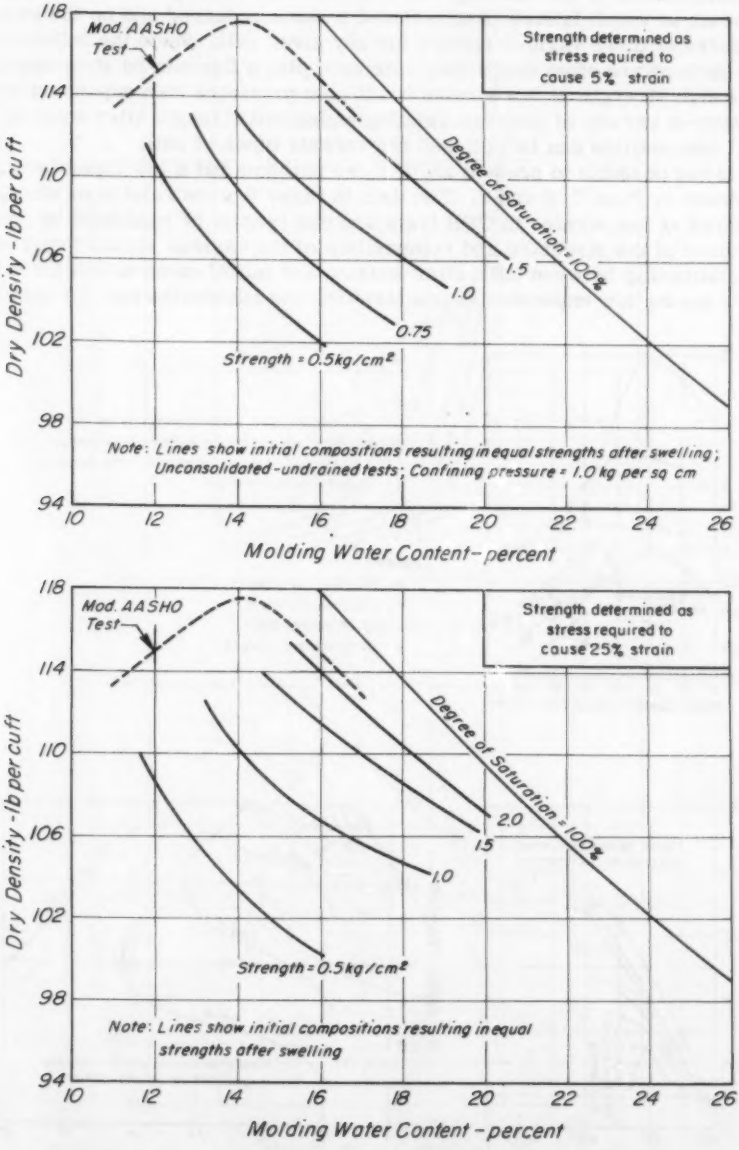

Mod. AASHO Test
Strength determined as stress required to cause 5% strain
Degree of Saturation = 100%
1.5
1.0
0.75
Strength = 0.5kg/cm²
Note: Lines show initial compositions resulting in equal strengths after swelling; Unconsolidated-undrained tests; Confining pressure = 1.0 kg per sq cm
Dry Density - lb per cu ft
Molding Water Content - percent
Mod. AASHO Test
Strength determined as stress required to cause 25% strain
Degree of Saturation = 100%
2.0
1.5
1.0
Strength = 0.5kg/cm²
Note: Lines show initial compositions resulting in equal strengths after swelling
Dry Density - lb per cu ft
Molding Water Content - percent

Fig. 6–RELATIONSHIP BETWEEN INITIAL COMPOSITION AND STRENGTH AFTER SWELLING UNDER SURCHARGE PRESSURE OF 1 PSI FOR SAMPLES OF SANDY CLAY PREPARED BY KNEADING COMPACTION.

structure of the soil (which also affects the pore-water pressures and is in turn determined by the molding water content and density at compaction), on the strain at which failure is considered to have occurred and on the relative influences of these various factors for any given soil. Since the influences of these factors are often conflicting—for example, a flocculated structure promotes high strength at low strains but it also promotes high expansion characteristics—a variety of patterns relating undrained strength after soaking to initial composition can be obtained for various types of soil.

It is not possible to present all of these patterns but a few typical examples are shown in Figs. 7, 8 and 9. The data in these figures relates to strengths measured at low strains in CBR tests and can readily be explained by consideration of the structure and composition of the various soils. Fig. 7 shows the relationship between CBR after soaking and initial composition for a lean clay(2) having low expansion characteristics and illustrates that for this type

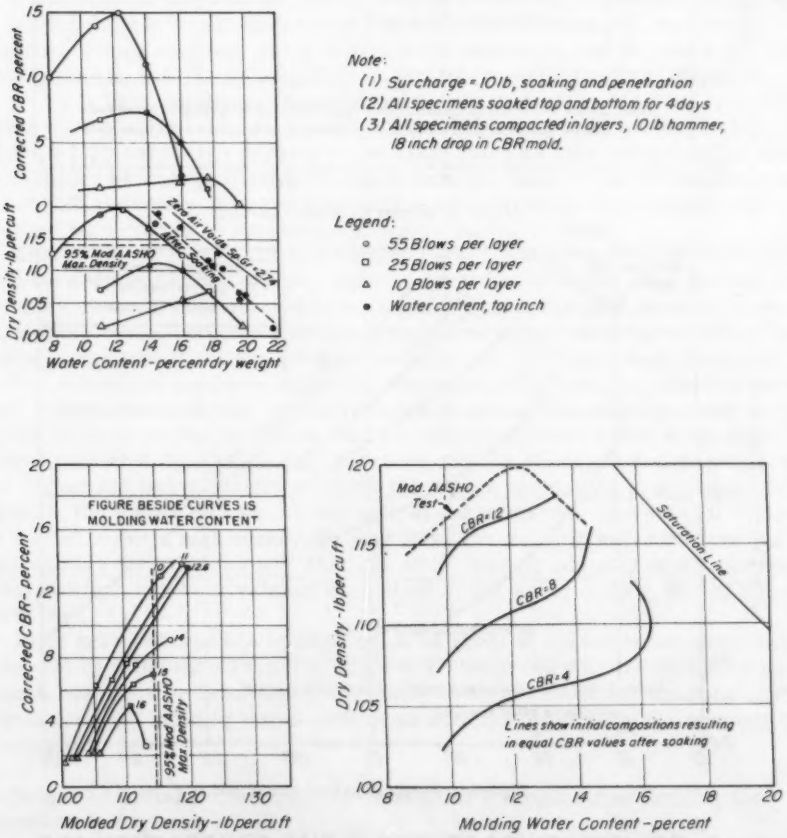
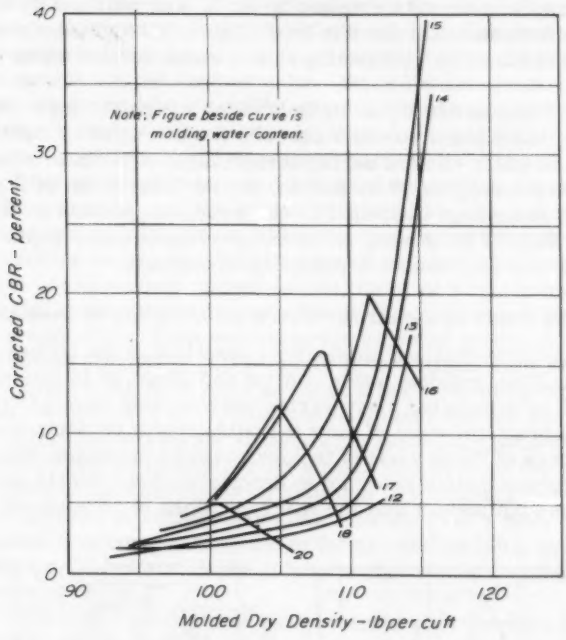

Corrected CBR-percent
Dry Density-lb per cuft
Water Content-percent dry weight
95% Mod AASHO Max. Density
Zero Air Voids Sp. Gr. = 2.7
After Soaking
Note:
(1) Surcharge = 10 lb, soaking and penetration
(2) All specimens soaked top and bottom for 4 days
(3) All specimens compacted in layers, 10 lb hammer, 18 inch drop in CBR mold.
Legend:
55 Blows per layer
25 Blows per layer
10 Blows per layer
Water content, top inch
FIGURE BESIDE CURVES IS MOLDING WATER CONTENT
Corrected CBR-percent
Molded Dry Density-lb per cuft
95% Mod AASHO Max. Density
Mod. AASHO Test
CBR=12
CBR=8
CBR=4
Saturation Line
Lines show initial compositions resulting in equal CBR values after soaking
Dry Density-lb per cuft
Molding Water Content-percent

Fig. 7 -RELATIONSHIP BETWEEN INITIAL COMPOSITION AND SOAKED CBR FOR SAMPLES OF LEAN CLAY.

(Data from Corps of Engineers Manual on Airfield Pavement Design)



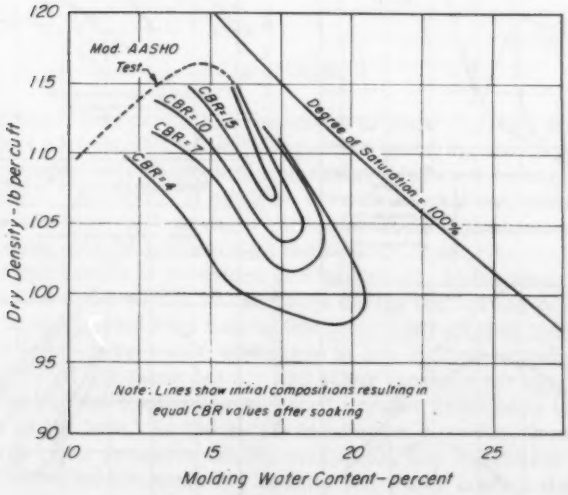


Fig. 8 – RELATIONSHIP BETWEEN INITIAL COMPOSITION AND SOAKED CBR FOR SAMPLES OF SILTY CLAY.

(Data after W.J Turnbull and C.R Foster)

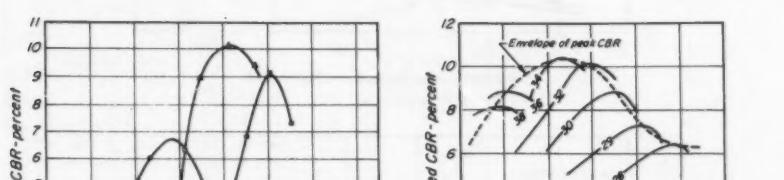
of soil, the highest strengths at low strains are customarily developed by the use of low water contents at compaction.

Fig. 8, showing similar data obtained by W. J. Turnbull and C. R. Foster[3] for a silty clay, indicates that for this type of soil, the highest strength at any given density is obtained by compacting it at a water content close to the line of optimums.

Finally, Fig. 9 shows data by J. R. Bell[4] for a highly plastic clay. For this type of soil, the highest strength can only be developed by careful control of dry density and water content during compaction. At a water content of 30 per cent and a dry density of 87 lb. per sq. ft., the CBR of the soil after soaking under a light surcharge is about 11. However, compaction to higher or lower densities than 87 lb. per sq. ft. or compaction at water contents higher or lower than 33 per cent causes a reduction in strength.

Importance of the Form of the Stress-Strain Relationship in Undrained Tests

It has been shown in this and an earlier paper[1] that the influence of method of compaction, molding water content and strain at failure on the undrained strength of compacted clays varies considerably with different types of soil. From a practical point of view it would clearly be desirable if the probable influences of these various factors could be predicted without the need for an elaborate testing program to determine their effects directly. Unfortunately it does not appear that the soil type along is an adequate basis for


Corrected CBR - percent
Legend
55 Blows per layer
25 Blows per layer
10 Blows per layer
Surcharge = 1.4 psi
Envelope of peak CBR
Note: Figures beside curves are molding water contents
Molded Dry Density - lb per cu ft

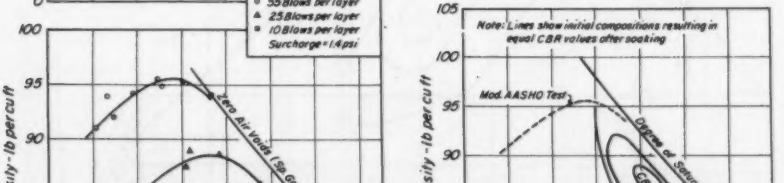

Dry Density - lb per cu ft
Zero Air Voids (Sp. Gr. = 2.81)
Note: Lines show initial compositions resulting in equal CBR values after soaking
Mod. AASHO Test
Degree of Saturation = 100%
CBR = 10
CBR = 6
CBR = 3
Molding Water Content - percent

Fig. 9 – RELATIONSHIP BETWEEN INITIAL COMPOSITION AND SOAKED CBR FOR SAMPLES OF HIGHLY PLASTIC CLAY.

(Data after J.R. Bell)

such predictions. Thus it would be helpful if a simple indicator test could be used for this purpose.

A consideration of the data reported indicates that the influence of method of compaction, molding water content and strain at failure on the undrained strength of compacted clays is determined largely by the form of the stress-strain relationships for the various soils. Since the stress-strain curves for samples compacted dry of optimum by any method of compaction are invariably steep initially and reach a maximum strength at relatively low strains, the major differences in general characteristics of soils are due to variations in the form of the stress-strain relationships for samples compacted wet of optimum. Thus, the shape of the stress-strain curve for a sample compacted wet of optimum by kneading compaction (which apparently produces as much change in structure as can possibly be caused) is a valuable indicator of the probable influence of method of compaction, molding water content and strain at failure on the relationship between undrained strength and the compacted condition of a clay soil. If the stress-strain curve of such a sample is initially steep and develops almost its full strength at low strain, then structural changes in the soil are not likely to have any appreciable effect on the strengths of samples, and the influence of molding water content for saturated specimens, method of compaction and strain at failure on the relationship between strength and compacted condition is likely to be small. However, if the stress-strain curve for a sample compacted wet of optimum is relatively flat and shows a progressive increase in resistance to deformation over a wide range of strain, then the method of compaction, strain at failure and molding water content for saturated specimens are likely to have a major effect on the strength characteristics of the soil.

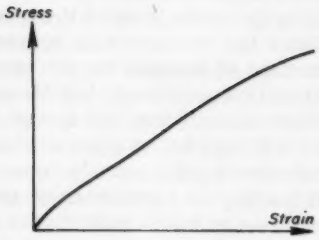
The influence of the form of the stress-strain relationships on soil strength characteristics, as indicated by the data presented earlier, is summarized in Table I.

CONCLUSION

Test data have been presented elsewhere to show that soil structure has a profound effect on a variety of soil properties including shrinkage, swell pressures, permeability, pore-water pressures induced during loading and undrained strength. However, it has also been found that soil structure appears to have little effect on soil strength characteristics expressed in terms of effective stresses, and its influence on the results of undrained strength tests on saturated specimens is therefore due essentially to its effect on the pore-water pressures developed in the samples during the testing process. Saturated samples of compacted clay having flocculated structures tend to develop substantially lower pore-water pressures at low strains in undrained tests than do samples of the same density and water contents having dispersed particle arrangements and consequently exhibit greater resistance to deformation at this stage of the test. As the strain increases, however, these initial differences in pore-water pressure tend to disappear, and specimens with different initial structures but the same dry density and water content therefore tend to have similar undrained strengths measured at high strains.

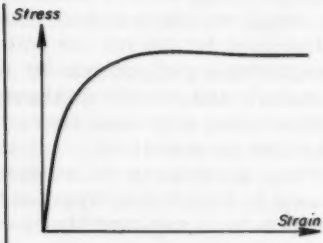
These facts have been utilized in the present paper to demonstrate and explain the different relationships which may exist between the initial composition and the undrained strength of compacted clays after soaking, both for

TABLE I.



If a sample compacted wet of optimum by kneading compaction has a stress-strain curve similar to that shown above:

1. The relationship between dry density, water content and strength as-compacted will vary greatly depending on the strain used to define the strength.
2. For samples prepared by kneading or impact compaction and strength determined at low strains, an increase in density at high degrees of saturation will cause a reduction in strength; if the strength is determined at high strains an increase in density may not cause such a decrease in strength.
3. If the strength is determined at low strains, the relationship between dry density, water content and strength as-compacted will be influenced greatly by the method of compaction; if the strength is determined at high strains, the method of compaction is not likely to have any appreciable effect on the form of the relationship between strength and composition.
4. For samples compacted to a given composition dry of optimum the method of compaction is not likely to have any appreciable effect on the strength.
5. For samples compacted to a given composition wet of optimum the method of compaction may have no appreciable effect on the strengths determined at high strains.
6. For samples compacted to a given composition wet of optimum and strengths determined at low strains, the strengths of samples prepared by different compaction methods will vary considerably and will increase in the order of the following compaction methods: kneading (least strength), impact, vibratory, static.
7. For samples prepared by kneading or impact compaction and soaked at constant volume the water content at compaction will have an appreciable effect on the final strength if the strength is determined at low strains (5 per cent); however, the water content at compaction may have no appreciable effect on the final strength if the strength is determined at high strains (say 20 per cent).
8. For expansive soils allowed to soak under a given surcharge pressure the relationship between strength after soaking and initial composition will vary widely depending on the method of compaction, surcharge pressure, the expansive nature of the soil and the strain at which failure is considered to occur.
9. For non-expansive soils compacted to a given density by kneading or impact methods and allowed to soak under a given surcharge, the strengths determined at low strains will be greater for those samples compacted at lower water contents; however, the strengths determined at high strains may not be influenced appreciably by the water content at compaction.



If a sample compacted wet of optimum by kneading has a stress-strain curve similar to that shown above:

1. The relationship between dry density, water content and strength as-compacted will vary only slightly whether strength is defined as the stress required to cause 5 per cent strain or 25 per cent strain.
2. Regardless of the definition of strength (so long as it is for strains greater than 5 per cent) the as-compacted strengths of samples prepared at a given water content by kneading compaction are not likely to show any appreciable decrease with an increase in density; however, with some soils a decrease in strength may in fact occur.
3. Regardless of the definition of strength (so long as it is for strains greater than 5 per cent) the as-compacted strengths of samples prepared at a given water content by any method of compaction are not likely to show any appreciable decrease with an increase in density. For those soils which do show such a decrease, the effect will be smaller for samples prepared by static compaction than for samples prepared by kneading or impact compaction.
4. For samples compacted to a given composition dry of optimum the method of compaction is not likely to have any appreciable effect on the strength.
5. For samples compacted to a given composition wet of optimum the method of compaction is not likely to have any appreciable effect on the strengths determined at high strains.
6. For samples compacted to a given composition wet of optimum and strengths determined at low strains, the strengths of samples prepared by different compaction methods will not differ appreciably and will increase slightly in the order of the following compaction methods: kneading (least strength), impact vibratory, static.
7. For samples soaked at constant volume the water content at compaction is not likely to have any appreciable effect on the strength though there may be some exceptions to this general rule.
8. For expansive soils allowed to soak under a given surcharge pressure the relationship between strength after soaking and initial composition will vary widely depending on the method of compaction, surcharge pressure and the expansive nature of the soil.

"strengths" determined at low and high values of strain. However, for samples soaked after compaction the problem is further complicated by the fact that the initial structure of a compacted sample also affects the amount of swelling, as well as the pore-water pressure induced by loading; furthermore, the final composition after swelling is also influenced by the surcharge pressure during the swelling process. Consequently a wide variety of relationships between initial composition and undrained strength after soaking can be found for various soil types and surcharge pressures. The forms of these relationships for samples soaked at constant volume, highly expansive soils and moderately expansive soils have been presented and discussed, and the significance of such structure-determining factors as molding water content and method of compaction on the results have been explained. The influence of the surcharge pressure on the form of the relationship between initial composition and undrained strength of soaked samples has also been illustrated.

Finally, the importance of the stress-strain relationship of soil samples prepared wet of optimum by kneading compaction, as an indicator of the probable significance of method of compaction, molding water content and strain at failure in determining the undrained strength characteristics of compacted soils has been discussed and the conclusions which might be drawn from a knowledge of this relationship have been summarized in Table I.

While it is hoped that the studies may have contributed to a better understanding of the subject, it must be stressed that much remains to be learned about this aspect of soil behavior. There is also a need for test data on a wide variety of soil types. In regard to this latter point, it is thought that much valuable information may well be hidden in the files of engineers concerned with the behavior of compacted clays. Furthermore, the authors have no doubt that exceptions can be found to some of the general comments made in the preceding pages. It is hoped, therefore, that the paper and its companion paper which presents many of the basic ideas utilized in the preceding pages, will encourage members of the Society to discuss these concepts and present supplementary data to the end that improved hypotheses may be developed. If this aim can be achieved, the temerity of the authors in attempting to rationalize a controversial subject, based on relatively limited available data, will have been justified.

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A REVIEW AND EVALUATION OF SOIL-CEMENT PAVEMENTS

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SUMMARY

The factors affecting the properties of soil-cement are reviewed. Quality and thickness design procedures for soil-cement base courses are critically examined. Field test section and airfield performance data are analyzed and compared with the performance of conventional flexible pavements.

I. INTRODUCTION

Since 1935 when the first controlled soil-cement construction was carried out near Johnsonville, South Carolina, soil-cement has been used on an increasing number of projects each year; at the present time several hundred million square yards of soil-cement roads and airfields have been built and are in service. In addition soil-cement has been used for such specialized purposes as for hardstands for jet aircraft parks,(29) car parks, and foundations for water storage tanks. Bryne and Holtz(3) report a further application in which a very wet mix known as "plastic soil-cement" was employed as a mortar for waterproof canal linings. The most widespread use of soil-cement, however, continues to be as a base course material for road and airfield pavements. Cement stabilization is now probably the most widely used and one of the most economical of the various methods of stabilization.

In addition to the uses cited above, several laboratory studies and field applications have been carried out in recent years using small quantities (less than 5%) of cement to modify the properties of soil.(43) The resulting product,

Note: Discussion open until May 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2294 is part of the copyrighted Journal of the Soil Mechanics Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 6, December, 1959.

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called cement-modified soil, has been found to possess less plasticity and greater water resistance than the untreated soil. The change in properties may be sufficient, in some cases, to make a satisfactory subbase course material of a normally unacceptable soil.

Since the construction of the Johnsonville road, a large number of laboratory and field studies have been carried out in an effort to obtain data on the nature, properties, and limitations of soil treatment with cement. Also, different organizations have established different procedures for the design of soil-cement mixtures. It is the purpose of this paper to summarize the present state of knowledge of the characteristics of soil-cement, to analyze the behavior of soil cement as indicated by available pavement service data, and to examine critically the presently used methods of soil-cement pavement and base course design.

II. Factors Affecting the Properties of Soil-Cement

Soil Type

The type of soil is the most important single factor affecting the quality of soil-cement. If the soil is unsuitable, little can be done at the present time to make the resulting soil-cement satisfactory. In general, experience has shown that soils meeting the following conditions can be hardened effectively through the addition of reasonable amounts of cement.(1,35)

Per cent finer than 0.002 mm - less than 35
Per cent passing No. 4 Sieve (4.76 mm) - greater than 55
Maximum size = 3 in.
Liquid limit less than 50%
Plasticity index less than 25%

The laboratory testing of a wide range of soils from many locations by Catton(5) indicated that grain size, density, water content, specific surface, organic content, void-cement ratio, and compressive strength of the untreated soil, all contribute to its response to cement stabilization. However, these factors are so diverse and interrelated in influence that none has a constant, major, predominating effect. In general, it was found that the amount of cement required for effective hardening increased with increasing silt and clay content, and that a soil may be acid, neutral, or alkaline and still respond well to treatment. A study by Sherwood(44) shows that the presence of calcium and magnesium sulfates in the soil or in the soil water can be very detrimental to the success of cement stabilization. Sulfate resistant cement was found to be no better in these materials than ordinary Portland cement. More recently work at MIT(20) has disclosed that the addition of sodium sulfate to the soil in conjunction with normal Portland cement is beneficial to early strength both as-cured and rewet. Although these results are not completely conclusive, since relatively few soils were studied and no durability tests were conducted, they do indicate that cation-anion associations must be considered in evaluating test results.

Based on the Public Roads soil classification system, the cement necessary for effective stabilization of various soils is as follows:

Soil Type		
Bureau of Public Roads Classification System	Unified Soil Classification System	Per Cent Cement by Volume
A-2 and A-3	GP, SP, and SF	6-10
A-4 and A-5	CL, ML, and MH	8-12
A-6 and A-7	CL and CH	10-14

In Table 1, typical compressive strength values are given for various soil types stabilized with about 10% cement by weight. Compressive strength values for untreated soils and concrete are listed for comparison. Values of strength and elastic properties of four soils studied by Felt and Abrams[15] are listed in Table 2. It should be emphasized that the values listed in Tables 1 and 2 are merely typical values. A particular soil may show large deviations from the average due to such factors as composition, type of cement used, method of cure, method of test, etc.

Investigations by Maclean[22] have indicated that the nature of the cation associated with the clay, as well as the type of clay mineral, influences the response of a soil to cement stabilization. It was found that calcium clays were the most easily stabilized, whereas sodium and hydrogen clays were

TABLE 1 - TYPICAL COMPRESSIVE STRENGTHS OF SOILS, CONCRETE, AND SOIL-CEMENT MIXTURES*

Material	Compressive Strength Range, psi
Untreated Soils:	
Clay, peat	less than 50
Well-compacted sandy clay	10 - 40
Well-compacted gravel, sand, clay mixtures	40 - 100
Soil-cement made with approx. 10% cement and:	
Clays, organic soils	less than 50
Silts, silty clays, very poorly graded sands, slightly organic soils	50 - 150
Silty clays, sandy clays, poorly graded sands and gravels	100 - 250
Silty sands, sandy clays, sands and gravels	250 - 500
Well-graded sand-clay or gravel-sand-clay mixtures and sands or gravels	500 - 1500**
Concrete	
Lean-mix concrete	500 - 2000
Concrete	2000 - 5000

*Data from reference 43.
**Soil-cement strengths up to 4000 psi have been recorded.

more difficult to stabilize. The addition of hydrated lime to sodium and hydrogen clays in order to convert them to the calcium form has resulted in satisfactory soil-cement. Experience has shown that soils composed of the non-expensive clay minerals are more suitable for cement stabilization than soils composed of the expanding-lattice type minerals.

A knowledge of pedological soil classification systems is sometimes helpful when considering soil-cement stabilization. Soils of the same texture, horizon, and series have been found by the Portland Cement Association to require about the same cement treatment.(38)

Organic Matter

A detailed study of the effect of organic matter on soil-cement was undertaken by Clare and Sherwood.(10) They found that organic compounds with high molecular weights, such as cellulose, starch, and lignin, did not affect strength; on the other hand, those with lower molecular weights, such as nucleic acid and dextrose, acted as hydration retarders and resulted in lower strengths. Since the strength in cement-treated surface soils was not found to be influenced by the total organic content, it is inferred that some active fraction of the organic material is responsible for hydration interference. Low strengths in treated surface soils were associated with a high capacity for the adsorption of calcium ions and a low pH. Work by the Military Engineering Experimental Establishment in England(30) has indicated that organic material deleterious to cement hardening occurs in the top organic layer of

TABLE 2 - ILLUSTRATIVE VALUES OF THE ELASTIC AND STRENGTH PROPERTIES OF SOIL-CEMENT MIXTURES*

Soil	Cement Content Per Cent		Values at 28 days, Moist Cure, psi			
	By Weight	By Volume	Comp. Str.	Mod. of Rupture	Mod. of Elast. Dynamic	Mod. of Elast. Static
Sand	3.8	5	450	110	2.05×10^6	
	6.0	8	800	180	2.75	
	8.5	11	1225	260	3.30	
Sandy loam	3.8	5	300	80	1.40	0.90×10^6
	6.1	8	650	145	2.00	1.25
	8.6	11	1025	215	2.60	1.65
Clayey Sand	5.7	7	475	105	1.30	
	8.3	10	625	150	1.50	
	11.0	13	800	195	1.75	
Silt Loam	8.0	9	525	125	0.90	0.55
	11.1	12	725	155	1.05	0.65
	14.2	15	900	190	1.25	0.75

Note: The lowest cement content listed for each soil is the quantity required to produce soil-cement that will meet wet-dry, freeze-thaw criteria for base course construction.

*Data from Reference 15.

podsol soil profiles. This material may occur under various types of vegetation and may extend to a depth of 11 feet or more.*

Cement

In the United States, Types I and IA (normal and air entraining, respectively) portland cements are generally used for soil-cement stabilization.(38) In Great Britain, considerable work has been done using "417" super-rapid-hardening cement and alumina cement as well as normal portland cement.(8) Super-rapid-hardening cement is similar to the U. S. Type III, high early-strength cement except that about 2% calcium chloride has been added. Although the ultimate strength obtained with rapid-set cements is about the same as, or slightly less than, that obtained using normal portland cement, much higher one- and seven-day strengths are sometimes achieved, particularly with sandy soils. In addition, the rapid-setting cements are sometimes effective in soils where normal cement is unsatisfactory, since the rapid-hardening cements contain greater amounts of calcium. Tests by Maclean(22) and Clare and Farrar(7) on the effect of cement fineness showed that in a sandy clay the one-day strength using a fine cement fraction equaled the seven-day strength using the unfractionated material.

Temperature

Investigations by Clare and Pollard(9) yielded several important facts concerning the effect of temperature on the strength of soil-cement mixtures.

1. The seven-day compressive strength increases with increasing curing temperature by 2% to 2-1/2% per degree when the temperature is near 25° C.
2. Soil-cement will harden in cold weather provided the temperature is above freezing.
3. If compressive strength is taken as the sole criterion of soil-cement quality, less cement is needed in warm weather than in cold weather.
4. Because of ambient temperature differences, soil-cement constructed during warm weather should be 50% to 100% stronger than similar construction made during cool weather, at least during the first three months of life of the construction.

Admixtures

The quality of soil-cement has been improved in many cases through the addition of suitable admixtures. Some poorly reacting, sandy surface soils can be used to produce a good quality soil-cement by adding 0.6% to 1.0% calcium chloride, sodium chloride, or sea water.(6,8,37) In some cases the addition of calcium chloride has resulted in a 50% reduction in cement requirement. However, the addition of calcium chloride to normally reacting soils causes little improvement.

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*Podsolic soils are developed under cool and damp climatic conditions providing an abundance of vegetative cover. The soils are acidic. Leaching is intense, and the organic and inorganic colloidal components are translocated from the upper horizons downward toward the water table.

The addition of asphalt to soil-cement has been found to decrease the strength in proportion to the amount of asphalt added.(24) At the same time, the use of 3% to 5% cement with 5% to 7-1/2% asphalt emulsion has been found to produce a product possessing both rigidity and water resistance.(43) Polyvinyl alcohol in quantities from 0.5% to 1.5% by weight of dry soil increased the strength of soil-cement up to 300%.

For the past several years research has been conducted at the Massachusetts Institute of Technology on the use of admixtures for the improvement of soil-cement properties. Among the more significant findings is that strength can be as much as doubled by means of low-level chemical treatment. A number of chemicals were shown to be effective in this respect, at least on some soils, but certain sodium compounds were found to result in the most marked strength increases and to have the widest range of applicability.(19,20) Sodium hydroxide was the most effective additive in heavy clays, sodium meta silicate in clean sands, and sodium sulfate and aluminates in lean clays and silts. Sodium sulfate was the best additive in soils containing organic matter. The use of strength-increasing additives at concentrations of 1% or less may prove economically feasible through savings in cement that may be effected to achieve a minimum strength and may permit the use of soils otherwise not suitable for stabilization. Likewise, for a given cement content it may be possible to increase strength sufficiently by the use of additives to effect a reduction in required base thickness.

III. Determination of Cement Requirement

In the United States the cement requirement for stabilization of a given soil usually is determined by a series of wet-dry and freeze-thaw tests on compacted specimens.(1,21,36,38) The general procedure is as follows:

1. Classify the soil and select several trial cement contents.
2. Prepare trial soil-cement mixes and determine the compaction characteristics.
3. Prepare two specimens from each trial mix at optimum water content, using standard AASHO compactive effort.
4. Subject one specimen to the wet-dry test (ASTM D559-44) and the other to the freeze-thaw test (ASTM D560-44).
5. Select the cement percentage by comparing the weight losses during the resistance tests with the allowable loss.

Detailed laboratory procedures are given in Reference 21 and 36. It is interesting to note that this method puts no requirement on the strength that must be obtained other than that the compressive strength must increase with age.

A recent investigation by Redus(40) on the condition of several soil-cement airfields has shown one case where the soil-cement disintegrated completely in the wet-dry test, but still performed satisfactorily under load. He also found two cases of soil-cement performing satisfactorily but failing the freeze-thaw test.

In Great Britain the cement requirement is determined on the basis of compressive strength. Specifications usually require a field compressive strength of 250 psi. It has been found that normal construction methods result in a field strength equal to about 60% of the laboratory strength for a given cement treatment. Therefore, the cement content is determined as that necessary to

give a laboratory compressive strength equal to 250 divided by 0.6, or about 420 psi. Samples are prepared at optimum moisture content and maximum density and cured at 100% relative humidity for 7 days prior to testing. Experience has shown that soil-cement meeting this compressive strength requirement generally will perform satisfactorily in the wet-dry and freeze-thaw tests. The CBR test has also been used for the laboratory evaluation of soil-cement.(22)

IV. Design of Soil-Cement Bases and Pavements

United States Practice

Most state highway departments base the quality design of soil-cement on the durability tests mentioned previously. California has somewhat more stringent requirements, specifying that the seven-day compressive strength shall be 650 psi.(4) Either procedure yields a base that is considerably stronger than an untreated granular base. Recognizing this fact, and considering the results of a nationwide survey of the various highway base course thicknesses in use, the Highway Research Board has recommended soil-cement thicknesses for various subgrade soils.(18) Generally, the same soil-cement thickness as granular base thickness is recommended on good quality subgrades, whereas a soil-cement thickness equal to three-fourths of the required thickness of granular base may be used on the weaker subgrades. It seems reasonable to suppose that this reduced thickness of soil-cement performs as well as a thicker layer of granular base because of its measurable flexural strength.

The California Highway Department determines the necessary thickness of soil-cement base on the basis of the resistance (R value) of the subgrade, the traffic intensity, the anticipated wheel loads, and the strength (C value) of the pavement material.(34) It is reported that using the California design method, the required thickness of soil-cement is generally 35-50% less than that required for a granular base. This procedure is notable in that it recognizes soil-cement as a construction material that can be designed for in the same manner as other common paving materials (asphalt, concrete).

British Practice

The Military Engineering Experimental Establishment has conducted investigations into the proper procedure for the design of soil-cement pavements. Maclean and Robinson(23) have pointed out that, for the high wheel loads and tire pressures now being used, the design must take into account the stresses imposed, the strength of the stabilized soil, and the strength of the subgrade. Since soil-cement possesses significant flexural strength, it was necessary to decide whether rigid or flexible pavement design methods should be used. On the basis of considerations described in the following paragraph, it was concluded that soil-cement behaves as a flexible pavement.(11,22,23, 25,26)*

The reasoning leading to this conclusion is essentially as follows. The flexural strength of soil-cement corresponding to an unconfined compressive

*Soil-cement has been considered a flexible material by the U. S. Corps of Engineers in their airfield evaluation studies.

strength of about 250 psi (which is the usual strength requirement for soil-cement pavements in Britain) is of the order of 50 psi. Using the Westergaard method of rigid pavement design, a 24-in. thickness of soil-cement would be needed on a subgrade having a subgrade modulus k of 100 psi per in. for a wheel load of 9,000 lb. In practice, however, a 6-in. thickness has been found adequate to carry this wheel load under these conditions. Therefore, flexural failures must have occurred causing the soil-cement to act as a flexible material.

The formation of closely spaced shrinkage cracks has been observed in almost all soil-cement construction. These cracks are usually fine so that interlocking between the two sides of the cracks remains high, causing the material to behave similarly to a crushed stone base. Measurements have shown that the CBR of a soil-cement base in which hair cracks have formed exceeded 300.

Maclean[22] has further suggested that compressive strengths much greater than 250 psi are probably undesirable, since beam action is more pronounced for higher strengths and flexural failure is associated with cracks occurring at greater spacing. When this happens the cracks are wider and interlocking is less effective. Data in support of this view are lacking, however, and the results of the pavement evaluation study summarized below indicate that in some cases a strength of 250 psi may be insufficient. Further, it does not seem unreasonable to expect that, should additives become available that permit the attainment of a very high-strength material (e.g., modulus of rupture = 200 psi) without increase in cement content, then flexural strength could be used in design with possible savings in thickness.

The Military Engineering Experimental Establishment has adopted the CBR method for forward airfield design procedure. However, the original Corps of Engineers design curves have been modified to specify a fewer number of coverages than were actually used to establish the curves. In general the results obtained using this method have proven satisfactory.

The British have also investigated the applicability of a method based on the shear strength of the subgrade. This procedure is applied to subgrades having a strength independent of the overburden pressure ($\phi = 0$).[25,27] In this method the shear strength of the subgrade soil is compared with the maximum shear stress induced at any depth by a given wheel load and tire pressure, as determined by the theory of elasticity. A thickness of soil-cement is selected such that at any depth greater than the base thickness the induced shear stresses are less than the shear strength of the subgrade. This method has proven satisfactory for the design of pavements over very weak subgrades.

V. Construction Methods and Equipment

It is not proposed to discuss construction methods and equipment in detail in this paper. However, whatever the construction method used (mix-in-place, central plant mix, or hand laying) and whatever the equipment used it is essential that the following criteria be met:

1. The proper water content for maximum density, uniformly mixed.
2. The proper cement content, uniformly mixed.
3. The attainment of some specified minimum density.

The importance of uniform mixing cannot be overemphasized. Baker[2] has found that the strength of soil-cement is proportional to the log of mixing uniformity; that is, the strength is a power function of mix uniformity. Thus, the effective stabilization of heavy clay soils probably is hindered more by the difficulty of mixing the stabilizing material with the soil than by any other factor. The development of the full potential of soil-cement requires considerable improvements in field mixing processes and equipment.

The cracking tendencies of soil-cement have been mentioned. This cracking necessitates some type of waterproof surfacing to prevent the downflow of water to, and the subsequent weakening of, the subgrade. A surface seal is also necessary during the soil-cement curing period to prevent excessive evaporation from the construction, but probably the most important function of the surfacing is to protect the soil-cement against the abrasive action of traffic. Unsurfaced soil-cement when subjected to intensive traffic tends to pit and ravel and soon becomes unsuitable for continued use. Bituminous wearing surfaces from 1/2 to 3 in. in thickness have been found satisfactory. In some cases a bituminous surface treatment is all that is needed.

VI. An Evaluation of the Performance of Soil-Cement Test Sections, Roads, and Airfield Facilities

The evaluation of the performance of soil-cement roads, airfields, and test sections which follows has been undertaken with the following objectives.

1. To illustrate the range of applicability of soil-cement stabilization and the generally successful results obtained by its use.
2. To illustrate those features peculiar to soil-cement that must be considered for satisfactory performance.
3. To compare the performance of soil-cement pavements with that of conventional flexible pavements.

Method of Analysis

The results obtained from nine soil-cement field test sections are listed in Table 3. The performance of the various test sections under the traffic shown is indicated by the number of coverages required to cause failure. The symbol ">" (greater than), column 13, indicates that no failure had occurred at the end of trafficking. One coverage is defined as one pass of the wheel load over any given small area of the section.

The point of failure for the test lanes listed in Table 3 was, in some cases, hard to define from the data given in the references. Insofar as possible, that number of coverages which resulted in sufficient permanent deformation and cracking to seriously hinder the movement of the test vehicles was selected. Failure may have occurred in one or both of two locations—subgrade and/or pavement. A subgrade failure results when the thickness of pavement is insufficient to prevent deformation in the subgrade. A pavement failure occurs when the soil-cement possesses insufficient strength to resist the repeated stressing and abrading action of traffic. As pointed out previously, fine shrinkage cracking is characteristic of soil-cement; therefore, the presence of these fine cracks is by no means indicative of pavement overstressing due to traffic. It has been found, however, that progressive widening of these cracks is a sign of overstressing, and that complete failure is incipient when they approach 1/8 in. in width.[27]

TABLE 3 — RESULTS OF SOIL-CEMENT FIELD TESTS

(1) Location	(2) Agency	(3) Ref. No.	(4) Surfacing	Soil-cement (5) Material Stabilized	(6) Cement Content, %	(7) Thickness in.	(8) Compressive Strength psi	Subgrade (9) Soil Type	(10) CBR
Fort Belvoir, Va.	ERDL	42	None	Clay gravel (GC) and sandy clay (CL)	7, 10, 11, 14	6	---	Red sandy clay	13
					7, 10, 11, 14	8	---	Red sandy clay	13
					14	6	---	Red sandy clay	4
					7, 11	6	---	Gray clay	4
					10, 14	6	---	Gray clay	4
					7, 11, 14	8	---	Gray clay	4
					10	8	---	Gray clay	4
					7, 10, 11, 14	6	---	Gray clay	
Hurn, Great Britain	MEXE	31	None	Sand	--	10	473	Sand	5
					--	6 (blocks)	186 (7D)*	Sand	6.4
					--	6	248 (7D)	Sand	5
Somerset, Great Britain	MEXE	33	None	Fines from crushed red sandstone (SF)	4-5	3	296	Clay	37
					4-5	6	257	Clay	10
					4-5	6	189	Clay	9
					4-5	9	151	Clay	11
					4-5	9	326	Clay	16
Somerset, Great Britain (reconstructed after tests described in ref 33)	MEXE	33	None	Fines from crushed red sandstone (SF)	4-4.75	12	239	Silty clay of medium plasticity	15
					4.6-5.7	12	315		15
					3.9-4.6	18	243		14
				Clay	14.1-16.9	18	547		16
				Crushed red sandstone	3.4-6.0	12	338		17
					3.6	6	277		16
					4.35-5.5	18	356		15
								12 in. of crushed rock on silty clay	
Brocksburst-Haddenheim, Great Britain	MEXE	28	PBS	Organic clay	26	13	92 (7D)	Clay	1
					25-1/2	18-1/2	85 (7D)	Clay	1
					26	25	74 (7D)	Clay	1
				Clay	--	23-1/2	308-408 (7D)	Clay	1
Dorset, Great Britain	MEXE	27	None	Clay (CH-CL)	--	6	318	Sandy clay	3.0
			None		--	6	301	Sandy clay	2.5
			PSP		17.9	6	219	Sandy clay	3.0
			PSP		17.9	6	258	Sandy clay	7.5
			None		20.0	7	215	Sandy clay	2.5
			None		15.4	13-1/2	221	Sandy clay	3.0
			None		16.2	13-1/2	231	Sandy clay	2.0
			None		14.3	13-1/2	268	Sandy clay	5.0
			None		14.4	18	294	Sandy clay	3.0
			None		16.1	18	241	Sandy clay	1.5
			None		---	18	---	Sandy clay	1.5
			None		15.6	18	219	Sandy clay	4.0
			None		16.2	18	252	Sandy clay	3.0
			None		14.3	13-1/2	198 (7D)	Sandy clay	6.0
			None		13.3	13-1/2	254	Sandy clay	2.5
			None		12.5	13-1/2	228 (7D)	Sandy clay	
Barksdale, La.	CE	13	3-in. AC	Selected loam	8.6	15	87 (flex.)	Medium plas clay	6
			3-in. AC		8.6	17	500 (comp)		6
Beltsville, Md.	CE	13	1-1/2-in. AC	Sand and clay mixture	10	6		Plastic cl	11+
					10	6	800 (28D)	Plastic cl	11+
Fargo, N. Dak.	CE	13	None	Gravelly sand	10	6		Coarse t	9
					10	6		fine sand	9
					10	6		plastic binder	9

* 7D and 28D = 7-day and 28-day strength. Other values at time of test.
** Assumes PSP = 9 in. equivalent.
+ Includes 1/4 in. of selected loam subbase.

(10) CBR	Traffic (11) Wheel Load kip	Traffic (12) Tire Pressure psi	Traffic (13) Cov. to Failure	(14) Total Pavement Thickness in.	(15) Flex. Pave. Thickness Required for Equal Performance in.	(16) Flex. Pave. Thickness Minus S/C Thickness for Equivalent Performance in.	(17) Remarks
13	15	100	60-200	6	6	0	Performances of the two types of pavement are roughly equivalent. Lack of data on the S/C quality and variation in subgrade CBR limit reliability of analysis.
13	15	100	60-200	8	6	-2	
13	6	70	>300	6	5	-1	
4	6	70	>300	6	8	2	
4	3	40	>2700	6	8	2	
4	6	70	>300	8	9	1	
4	3	40	>201	8	5-1/2	-2-1/2	
4	4.5	40	>200	6	9	3	
5	50	110	>200	10	>22	>12	CBR values appear inordinately low for type of subgrade used. Mean CBR of an adjacent PRP test panel increased from an initial 4.4 to 10.6% after traffic.
6.4	50	110	>200	6	>18	>12	
5	25	160	50	6	12-1/2	6-1/2	
37	12.5	110	>150	3	2-1/2	-1/2	CBR values varied widely over site. Computed flexible pavement thicknesses could be in considerable error. Base failure under 160-psi tire. Subgrade and base failures in other sections.
10	12.5	110	38	6	5-1/2	-1/2	
9	25	160	7	6	6	0	
11	12.5	60	106	9	6	-3	
16	12.5	110	127	9	4-6	-3 to -5	
15	12.5	160	>200	12	—	S/C over designed	CBR values determined using cone penetrometer. Converted to CBR using 55 cone index = 1 CBR. Results appear inconsistent if subgrade failure is assumed. Favorable performance of clay-cement suggests that failures may have been in stabilized sand.
15	25	110	100	12	7	-5	
14	25	160	168	18	9	-9	
16	25	160	>550	18	>9	>-9	
17	50	110	40	12	7	-5	
16	50	110	150	18	10	-8	
15	50	110	40	18	8	-10	
1	2.5	80	160	13	11	Surface failure, low compressive strength	Organic clay difficult to stabilize with cement.
1	2.5	80	160	18-1/2	11		
1	2.5	80	160	25	11		
1	2.5	80	>37,500	23-1/2	21	-2-1/2	
3.0	25	110	6	6	10	4	Data indicate a thickness reduction of 9-6 in., in general, for S/C as opposed to flexible pavement. The lower the CBR, the greater the thickness reduction.
3.5	12.5	60	75	6	12	6	
2.5	12.5	60	>200	15**	13-1/2	1/2	
3.0	25	110	50	15**	16	1	
7.5	12.5	60	150	7	8-1/2	1-1/2	
2.5	25	60	95	13-1/2	19-1/2	6	
3.0	25	110	110	13-1/2	19	5-1/2	
2.0	12.5	110	>200	13-1/2	18	4-1/2	
5.0	25	110	>200	18	16	-2	
3.0	50	110	150	18	28	10	
1.5	25	160	>200	18	30	12	
1.5	50	60	>200	18	42	24	
4.0	50	60	>200	18	>24	>6	
3.0	25	110	150	13-1/2	20	6-1/2	
6.0	12.5	110	>200	13-1/2	9-1/2	-4	
2.5	12.5	160	130	13-1/2	15	1-1/2	Base failure along construction joint.
6	20	50	>5000	21+	19	-2	S/C of inferior quality. Contained clay balls to 4% of volume. S/C performed equally as well as other flexible bases tested at Barksdale.
6	50	50	400	24+	23	-1	
11+	15	40	227	7-1/2	7-1/2	0	Trafficked next to construction joint.
11+	15	40	2682	7-1/2	11	3-1/2	Trafficked in interior of slab.
9	30	55	120	6	11	5	Failures commenced at edges and construction joints then progressed inwards. Failures attributed to both S/C at edges and joints and to subgrade deformation.
9	20	38	530	6	11	5	
9	12.5	38	1470	6	10	4	

The thicknesses of flexible pavement construction required to give a performance equal to that of the soil-cement test sections are also listed in Table 3. These flexible pavement thicknesses were determined from plots prepared from the CBR design curves now in use by the Corps of Engineers. The plots permitted determination of the required flexible pavement thickness for any specified number of coverages. Account was taken of tire pressures and number of coverages in addition to wheel loads. The differences between the actual soil-cement pavement thickness (including wearing surface, if any) used and the flexible pavement thickness required for equal performance are listed in Table 3, column 16. The field tests are analyzed individually in the following paragraphs.

Performance of Field Test Sections

ERDL Test Sections.—The Engineer Research and Development Laboratory, Ft. Belvoir, Va., test sections[42] were constructed in an effort to help establish a yardstick for the load-carrying capacity of cement treated surfaces. The test panels were constructed over two different subgrades; a red sandy clay and a gray clay having average CBR values of 13 and 4, respectively. Soil-cement surfaces of 6- and 8-in. thicknesses were constructed using a clay gravel (GC) and a sandy clay (CL) with cement contents ranging from 7% to 14%. The completed construction was cured for three winter months under a blanket of loose soil. No data are given relative to the quality of the stabilized material. Trafficking was carried out using, successively, 100 passes of a 1/4-ton jeep, 100 passes of a 3/4-ton truck, 900 passes of a 2-1/2-ton 6 x 6 truck, 100 passes of a 4-ton 6 x 6 truck, 100 passes of a 6-ton 6 x 6 truck, and 200 passes of a 15,000 lb. wheel load. Because of this successive trafficking with different vehicles, the test results are difficult to interpret. The listing of wheel loads and coverages in Table 3 is based upon the best estimate of performance that could be made from the description of the test section behavior under traffic as given in Reference 42. The comparison between the thickness of the soil-cement panels and the thicknesses of flexible pavement for similar performance shows approximately equal thicknesses required.

Hurn, Great Britain, Test Sections.—The Hurn test panels[31] were composed of a sand-cement pavement over a sand subgrade. On the basis of the CBR values reported, a flexible pavement about twice as thick as the soil-cement pavement would be needed for an equivalent performance. The soil-cement compressive strengths are not exceptionally high; therefore, it is doubtful that there was much rigid pavement action. The subgrade CBR values appear inordinately low for a granular material. Tests on a panel adjacent to the soil-cement sections indicated that trafficking increased the CBR under pierced steel plank from 4.4 to 10.6. In all probability the CBR of the sand under the soil-cement also increased with trafficking. It seems reasonable to conclude therefore that the subgrade was actually considerably stronger than indicated by the CBR values.

Somerset Great Britain Test Sections.—The Somerset test sections[32] were constructed using a base course of 4-5% cement mixed with the fines from a crushed red sandstone (SP classification) over a heavy clay subgrade. Subgrade CBR values varied widely over the site; those listed in Table 3 are the approximate values for the subgrade under the failed sections of pavement. Further testing of the subgrade after the completion of testing indicated that the CBR decreased with depth; thus the effective CBR values were probably

slightly less than those listed in the table. A comparison of the actual soil-cement thickness with the required thickness of flexible pavement for equivalent performance indicates that the soil-cement was slightly inferior to the gravel base in a normal flexible pavement. In view of the uncertainties in the CBR values this difference is not felt to be significant.

Reconstructed Somerset Test Section

After steps had been taken to equalize subgrade conditions over the entire Somerset test area and a thorough investigation of subgrade strength variations with depth had been made, test panels were reconstructed.(33) Soil-cement pavements of 12- and 18-in. thicknesses were constructed using the quarry fines from the same red sandstone. In addition, one 18-in.-thick soil-cement panel, using a heavy clay stabilized with 14-19% cement, and one panel consisting of 6 in. of stabilized crushed sandstone over 12 in. of compacted crushed sandstone were constructed. CBR values decreased from 15-25 at the surface of the subgrade to 8-15 at a point 3 ft below the surface. Values listed in Table 3 are the average of the surface CBR and the mean CBR over a 3-ft depth. All test panels with the exception of the one composed of 18 in. of clay-cement and one of sand-cement that was subjected to only a 12,500-lb wheel load, failed under the imposed traffic. A flexible pavement of from 3 to 10 in. less in thickness (depending on wheel load and tire pressure) would have given the same performance. An 18-in. pavement of a 15-CBR subgrade should be capable of sustaining over 2,000 coverages of a 50,000-lb wheel load without distress. In view of this and in view of the fact that the clay-cement pavement successfully carried the 25,000-lb wheel load, whereas the sand-cement failed under this wheel load, it seems most likely that failure occurred in the sand-cement pavement itself. The compressive strengths of all panels of stabilized sand were well over 200 psi at the time of trafficking. Apparently this compressive strength is no guarantee of satisfactory performance, at least in granular materials.

Brockenhurst-Haddenheim Test Road, Great Britain

The Brockenhurst-Haddenheim test road(28) was constructed over a very weak subgrade having a CBR of 2-1/2 at the start of the test and of less than 1 at the completion of the test. Soil-cement pavements of 13-25 in. in thickness were constructed using an organic clay in some sections and an inorganic clay in others. Trafficking was done with trucks specially loaded to give 2,500-lb wheel loads. All sections constructed of organic clay failed by 160 coverages. The failure was attributed to the very low compressive strength of the stabilized material. The seven-day compressive strength of the clay stabilized with 26% cement ranged from 74 to 92 psi. The inorganic clay-cement sections, having seven-day compressive strengths of 308-408 psi, were in excellent condition after 37,500 coverages with the 2,500-lb wheel load. Analysis of wheel load and subgrade conditions indicates that a flexible pavement 2-1/2 in. thinner than that of the soil-cement actually used would provide equal performance. It should be noted that the Corps of Engineers CBR design curves do not normally extend below CBR values of 3. Comparative values for required flexible pavement thickness were determined by the method of Turnbull and Ahlvin.(46)

Dorset, Great Britain Test Panels

Test panels were constructed at Dorset[27] using a 12.5-20% cement with a heavy inorganic clay over a very weak sandy clay subgrade (CBR 1.5 to 7.5). Pavements of 6-, 7-, 13-1/2, and 18-in. thickness were used. Two additional panels were composed of 6 in. of clay-cement overlain by pierced steel plank (PSP). For purpose of analysis, the PSP is assumed to be the equivalent of 9 in. of flexible pavement, based on the results of Corps of Engineers landing mat studies. In this test section, traffic was continued only until 200 coverages were made, as this meets the British forward airfield requirements. All failures occurring prior to 200 coverages were believed to be subgrade failures. The comparison of the thickness of the soil-cement test sections with Corps of Engineers flexible pavement thickness requirements for equal performance shows that for CBR values greater than 3 the two types are roughly equivalent; however, the loads usually were supported by the soil-cement with slightly less thickness than that indicated by the flexible pavement design curves. At CBR values less than 3, the use of soil-cement resulted in significant reductions in thickness over that required by the CBR method, with the greatest differences at the lowest CBR's.

It seems possible that over such weak subgrades the flexural strength of soil-cement is effective in carrying the wheel loads. Very weak subgrades usually have a high water content and as a result the bottom portion of a thick soil-cement layer would not be expected to undergo appreciable drying. Therefore, shrinkage cracking is not likely to occur to as great an extent in the lower portion of the soil-cement as in the top portion, and slab action could be significant. Examination of the soil-cement base at the conclusion of the Barksdale accelerated traffic tests[12] indicated that the material was fractured throughout, both in areas affected by traffic and in areas outside the traffic lane, into numerous small, interlocking fragments. However, the fractures were more numerous in the top 6 in. than in the bottom of the layer. Rough computations of required pavement thicknesses on subgrades with CBR's less than 3, using Reissner's theory[41] of flexible surfaces over elastic subgrades, show reasonable agreement with the test data. Also, the Military Engineering Experimental Establishment[27] found that the shear strength method of design applied to the Dorset sections gave good results.

Barksdale, La. Tests

Base courses of soil-cement were included in the Barksdale Field service behavior tests conducted by the Corps of Engineers for the purpose of determining design requirements for flexible pavements under heavy wheel loads.[12] Two soil-cement sections were constructed; one was trafficked by a 50,000-lb, single-wheel load and the other by a 20,000-lb, single-wheel load. Each section was constructed as a single slab 45 ft. wide and 75 ft long over a 4-in. thick subbase of selected loam. The soil-cement base varied uniformly in thickness from 10 in. to 17 in. from one end to the other. Surfacing consisted of 3 in. of asphaltic concrete. Thus total pavement thicknesses varied from 17 in. to 24 in. The subgrade was a clay of medium plasticity with a CBR of 6.

The base course was constructed of a selected loam to which about 8.6% cement was added. This cement content was lower than intended and the resulting material was reported to be of inferior quality, although the compressive strength was of the order of 500 psi and the flexural strength about

90 psi for samples taken outside the traffic lane at the conclusion of trafficking. It was noted that about 4% of the volume consisted of balls of untreated clay.

By 100 coverages with the 50,000-lb. wheel load the asphaltic concrete had moved about 2 in. in the direction of travel. By 400 coverages the pavement had failed completely from shear deformation of the subgrade, subbase and soil-cement. After 5,000 coverages with the 20,000-lb. wheel load, failures had progressed from the thin end of the section to a point where the total pavement thickness was about 21 in. Subgrade deformation was the primary cause of failure in the thin end of the section. As with the 50,000-lb. section, the bond between the asphaltic concrete and the soil-cement was insufficient to prevent movement of the pavement due to traffic. Table 3 shows that flexible pavement from 1 to 2 in. less thick should give a performance equivalent to the soil-cement sections. However, the performance of these soil-cement sections was equally as good as that of the other flexible bases tested at Barksdale; i.e., the service behavior tests on conventional bases indicated required thicknesses slightly greater than those given by the CBR curves.

Beltsville, Md., Traffic Tests

The Beltsville accelerated traffic tests[13] were conducted on an existing pavement with a 15,000-lb. wheel load. The base which had a 28-day compressive strength of 800 psi consisted of 6 in. of a sand and clay mixture stabilized with 10% cement. The soil-cement was surfaced with 1-1/2 in. of asphaltic concrete of poor quality. The subgrade was a plastic clay (CL). The best estimate of the subgrade CBR is about 11. Two test lanes were trafficked; one adjacent to a construction joint and the other across the interior of the slab. Failure of the entire lane adjacent to the construction joint occurred after 227 coverages. The data indicate that the failure was caused by shear deformation and consolidation in the subgrade, as a result of the weakness of the soil-cement at the construction joint. Failure of the interior traffic lane occurred in two local areas by 2,682 coverages. Indications were that these failures were caused by detrimental shear and consolidation in the subgrade and the weakening effect of test pits adjacent to the traffic lane. These results indicate that construction joints may be critical points in a soil-cement base. At the same time, however, the results of plate bearing tests indicated that the subgrade under the lane adjacent to the construction joint was considerably weaker than the subgrade under the interior lane. Comparison of the soil-cement performance with flexible pavement performance, Table 3, indicates equivalence in the case of the soil-cement adjacent to the construction joint, but superiority of the soil-cement in interior sections. Uncertainties in the analysis are introduced, however, due to lack of reliable subgrade CBR data.

Fargo, N. D., Traffic Tests

Accelerated traffic tests were conducted on an existing runway at the Fargo, North Dakota, Municipal Airport using a 15-cu-yd scraper which could be loaded to give wheel loads of 30,000, 20,000, and 12,500 lb.[13] The base consisted of 6 in. of a gravelly sand stabilized with 10% cement over a subbase of a prepared mixture of coarse to fine sand with 25-30% silty clay as a binder. The subbase tapered from a thickness of 12 in. at the runway edges to 24 in. at the center line of the runway and had a CBR of 9. The highly plastic fat clay subgrade had a CBR of only 3. No surfacing was used over the soil-cement.

Failures commenced at the edges and construction joints and progressed towards the center. The failures in all three tests were attributed to a combination of weakness of the soil-cement at construction joints and edges and to excessive deflection caused by elastic movement, consolidation, and shear deformation in the subbase and subgrade. The data in Table 3 indicate that the soil-cement thickness was 4-5 in. less than the required flexible pavement thickness over a subbase CBR of 9. In addition, the combined thickness of soil-cement and subbase from the runway edge to about one-fourth of the distance to the center line was less than required by the CBR curves for a subgrade CBR of 3 for both the 30,000 and 20,000-lb wheel loads for the number of coverages sustained.

Performance of Soil-Cement Roads

Few data are available in sufficient detail to permit a comprehensive evaluation of the performance of soil-cement roads in service. However, for those cases where design, traffic, and performance data have been obtained, the results are encouraging.(17,18,22,45) In road construction, soil-cement has been used almost exclusively as a base course material with some type of bituminous wearing surface, ranging from surface treatment to 2 or 3 in. of asphaltic concrete. The behavior of soil-cement road bases indicates that with reasonably good subgrade conditions a 6-in. thickness of base is adequate. Cement contents ranging from 3.5% to 14% have been used. As previously mentioned, the Highway Research Board recommends a soil-cement thickness of three-fourths of the flexible base thickness required over weak subgrades, but an equal thickness of soil-cement and flexible base over the stronger subgrades.

Performance of Soil-Cement Airfield Facilities

Table 4 presents pertinent data concerning the pavement, soil-cement base, subgrade, traffic, and performance of 35 airfields on which soil-cement was used for at least some of the facilities. These data were obtained from Corps of Engineers airfield evaluation reports, most of which are dated 1944 and give the construction, traffic, and performance data to that date. With the exceptions listed in Note 1 of Table 4, the quality of the soil-cement was not evaluated, but the material was arbitrarily assigned a CBR of 50 or 80 based on its service behavior. In those cases where the soil-cement CBR was determined, it exceeded 80. The method used for computing the coverage data is given in Note 3 of Table 4.

The materials stabilized and the subgrades for the various airfields range from lean clays through gravels. A 6-in. thickness of soil-cement was used in most cases. Cement contents averaged about 10%. Surfacing varied from none to 4 in. of asphaltic concrete.

The data in Table 4 reveal several points of interest. Shrinkage cracks appear common to many of the bases, but they definitely do not indicate failure. In all cases where a bituminous surfacing was not used the soil-cement showed excessive abrasion, pitting, rutting and raveling. Columbia, Santa Maria, and Victorville Airfields offer a good comparison between surfaced and unsurfaced behavior. On these three fields, surfaced soil-cement performed satisfactorily, whereas the same material unsurfaced was excessively worn. These results show rather conclusively that unsurfaced soil-cement is not capable of withstanding the abrasive action of traffic.

There were no subgrade failures, with the exception of that caused by fill settlement at Cochran Field, in those cases where the actual pavement thickness was greater than that required by flexible pavement design considerations. Of 11 airfields where the thickness of soil-cement plus surfacing was less than that required for flexible pavement by CBR design for the given traffic conditions, 6 were performing in a satisfactory manner and 5 had subgrade failures or showed definite signs of overstress.

VII. CONCLUSIONS

The following conclusions relative to the properties, design and performance of soil-cement pavements are warranted.

1. A wide range of soil types may be stabilized with cement. Such factors as temperature, organic matter, type of clay mineral, and type of cement may influence the strength of the resulting material.
2. The use of admixtures for increasing the strength and durability to soil-cement is promising.
3. Shrinkage cracking of soil-cement bases is to be expected, but these cracks are not signs of failure. A bituminous wearing surface of some type is needed over soil-cement to protect it from abrasion and to keep water out of the shrinkage cracks.
4. Construction joints and pavement edges are likely to be critical points in soil-cement bases, possibly due to inferior mixing of the cement with the soil in these areas.
5. The subgrade protection provided by soil-cement pavements as currently designed and constructed is about the same as that to be expected from an equal thickness of conventional flexible pavement construction, for subgrades having CBR values greater than 3. Of the total of 85 test panels and airfields listed in Tables 3 and 4, 42 gave a performance equal to that expected of the same or a greater thickness of flexible pavement, 19 gave a poorer performance than to be expected from an equal thickness of flexible pavement; 7 cannot be compared due to lack of data; 3 cannot be compared because the soil-cement was of inferior quality; and 14 cannot be compared because the soil-cement was significantly thicker than required for the imposed traffic. For very weak subgrades (CBR less than 3) the thickness requirement for soil-cement is generally less than that for flexible pavements according to the CBR design curves. This may be because either the CBR method is too conservative on such weak subgrades or thick soil-cement pavements over weak subgrades contribute slab action.
6. A more satisfactory method for the quality design of soil-cement is needed. Of the 38 bases which were thick enough to prevent failure in the subgrade 13 were not of sufficient quality to prevent failure within the base itself. As indicated by the data in Table 3 the compressive strength may not be a suitable measure of soil-cement quality, particularly for sand-cement. For example, the compressive strengths of the failed sand-cement pavements in the reconstructed Somerset test sections ranged from 243 to 356 psi, whereas the strengths of the clay-cement pavements at Dorset, where failures were in the subgrade rather than in the base, ranged from 198 to 315, or slightly less than at Somerset. On the other hand, design on the basis of durability tests may be

TABLE 4 — PERFORMANCE OF CEMENT-STABILIZED AIRFIELD PAVEMENTS AND BASE COURSES
(Data from Corps of Engineers Airfield Evaluation Reports)

Field	Location	Soil-cement Facilities	Pavement		Soil-cement Base		
			Type	Thickness in.	Material Stabilized	Thickness in.	Cement Content %
Aiken	Aiken, S.C.	Taxiways, hardstands	Rt-2 prime sand-tar seal	-----	Sand clay	4	10
Anderson	Anderson, S. C.	All	AC	1-1/2	Sandy clayey silt	6	11
Atlanta	Atlanta, Ga.	4 runways Most taxiways Parking area	AC	1-1/2	Cecil clay, micaceous sandy loam	6	10-12
Avon Park	Avon Park, Fla.	All	Surface tar AC	----- 1-3	Silty sand (SM)	6	12-18
Biggs	El Paso, Tex.	All runways Some taxiways	AC	2	Gravelly sandy silt (SM)	6	10-12
Blackland	Waco, Tex.	Runways N-S, NE-SW Pads at apron Taxiways G, X, and Y	AC	2	Clayey gravel (GC-CL)	6-8	3-4
Chatham	Savannah, Ga.	Runway shoulders	Sand-tar seal	-----	(SP)	6	12
Cochran	Macon, Ga.	Aprons Taxiways F, G, H, K, L, N Runway shoulders	AC	1-1/2	Sandy loam, clayey loam (SC)	6	9
Columbia	Columbia, S.C.	Taxiways Hardstands	AC None	1-1/2 -----	Clayey sand (SW-SC)	6	7
Concord	Concord, Calif.	Runways NE-SW, NW-SE, taxiways	AC	2-4	Sandstone	5-6	3-5
Dale Mabry	Tallahassee, Fla.	Apron E-W runway	None Bituminous surface treatment 1943	----- 1	Sand	6	10
Foster Aux. 4	Victoria, Tex.	Runways N-S, NW-SE, E-W, taxiway to N-S runway	Seal Coat	-----	Bank run sandy gravel (GM-GC)	6	7-9
Grand Central	Glendale, Calif.	Runway extension, runway widening, hardstands	None	-----	Sandy gravel	6	13
Harding	Baton Rouge, La.	Apron (west side), east taxiway	Sand-Asphalt	2	Sandy silt (MH)	6	10
Herbert Swart	Macon, Ga.	All	AC	1-1/4	Sandy clay (SC-CL)	6	10
Houlton	Houlton, Maine	Apron N, S, center taxiways	AC	1-1/2	Silty gravelly sand (GW)	7	12, 1
Lomita Flight Strip	Lomita, Calif.	Runway	AC	4	(SM) nonplastic	6	1
Long Beach	Long Beach, Calif.	Anchorage shoulders	None	-----	Decomposed granite	6	1

(Continued)

Subgrade Soil Type	CBR	Traffic Coverages	Traffic Wheel Load kip	Flexible Pavement Thickness Required for Given Traffic in.	Total Pavement Thickness used in.	Performance (To Time of Evaluation, 1944)
4-in. sand clay Sandy loam (CL)	24.5 18	400	15	5	4	Good condition; many cracks in S/C, but no failures.
Sandy clayey silt (MH)	5.7-14.7	-----	--	-----	7-1/2	Good condition, no appreciable wear.
(MH, OL)	10	20,000	15 (est)	11-1/2	7-1/2	Good condition, some cracking in S/C.
Uniform fine to medium sand (SP)	26	2,000	30	7-1/2	7-8	Condition of areas ranged from fair to excellent.
Gravelly sandy silt (SM,SC)	8-47 26 (avg)	12,000	30	6	8	Good condition, cracks in S/C gradually extending up through AC surface.
4-6in. subbase Clayey sandy silt (CL)	24-100 7	1,200	3	3	8-10	Good condition, clay-gravel base had failed prior to addition of cement. Cement modified soil.
Fine sand (SP)	37	-----	--	-----	6	Good condition.
Sandy loam, clayey loam (SC)	17	10,000	2	4	7-1/2	Some failures due to underlying fill settlement. Repaired prior to surfacing with AC. S/C cracked but in good condition from then on.
Fine sand, some clay (SW, SC)	60	11,000	17.5	2-1/2+	6 to 7-1/2	Fair where unsurfaced, good where surfaced. Usual S/C cracks. Some sealing needed.
31 to 42-1/2-in. fill (CL, ML, SW, CH, SC)	10-80 5-9	210	7	5	7-10	Good condition, usual S/C cracks.
Medium and fine sand (SM)	25	1,100	30	7	6-7	S/C not evaluated. Presumed in good condition.
5-6 in. gravel subbase Clayey sand	15 5	1,700	3	3	6	Good condition. Needs surfacing.
Silty sand (SM)	35	400	30	3-1/2	6	Fair to good condition. Resurfaced after S/C began to pit and ravel.
Silty clay (CL)	8	600	15	10-1/2	8	Subgrade failure. Constant heavy maintenance required. Progressive failures began with start of 15-kip wheel load traffic.
(SC)	10	30	30	8	7-1/4	Good condition.
Silty gravelly sand (GW)	30 Normal 6 Spring	220	22	4 Normal 13-1/2 Spring	8-1/2	Pavement surface uneven and considerably cracked.
24-in. subbase (ML, CL)	50 3	600	7	1-1/2	10	Good condition.
(CL)	5-20	5,000 (est)	22	8-1/2 to 22	6	Surface scaling, some broken pavement. Suitable for occasional use only. Should have AC surface.

(Continued)

TABLE 4 — PERFORMANCE OF CEMENT-STABILIZED AIRFIELD PAVEMENTS AND BASE COURSES (Continued)
(Data from Corps of Engineers Airfield Evaluation Reports)

Field	Location	Soil-cement Facilities	Pavement		Soil-cement Base		
			Type	Thickness in.	Material Stabilized	Thickness in.	Cement Content %
March	Riverside, Calif.	Extension, runways 2, 3 Anchorage landing mat	AC	1-1/2	(GC, SC)	6	12
Moody	Valdosta, Ga.	N-S runway N-S taxiway	AC (1952) Seal	1-1/2 1/8	Clayey sand (SC)	8	8
Morris	Charlotte, N.C.	Apron 2-3 runway shoulders	AC	1-1/2 2-1/2	Mica schist	4-1/2 to 6	7
Muroc	Muroc, Calif.	N circle drive S circle drive	AC (later)	3	Sandy clay	6	10 (?)
Myrtle Beach	Myrtle Beach, S.C.	Runway	Sand-asphalt	2	Sand (SP-SM)	6	11
Ocean Drive Flight Strip	Ocean, S.C.	All	Sand-asphalt	3	Shell sand (Coquina)	6	8
Orange County	Santa Ana, Calif.	NE-SW runway Shoulders Taxiways	AC	1-1/2	Silty sandy gravel (SM)	6	8, 3
Palm Beach Intl.	W. Palm Beach, Fla.	Runway widening and old base, apron C	AC	3-1/2 4-1/2	Shell sand (GP)	6	12-18
Pampa Aux. 2	Pampa, Tex.	Runways NE-SW, NW-SE All shoulders	AC	1-1/2	Sand-gravel (SW)	5	4
Reno	Reno, Nev.	Apron	AC	2-1/2	Gravel	6-1/2	12
Robins	Wellston, Ga.	Hardstandings, stubs	Seal coat	-----	Sand-clay, gravel-sand-clay. Poorly graded	6	10
Roswell	Roswell, N. Mex.	Half of parking apron	AC	1-1/2	Sandy gravel (GM)	6	10
Santa Maria	Santa Maria, Calif.	Extension runway 3 Taxiway 1 B	None	-----	Silty sand (SC)	6	9
Shaw	Sumpter, S.C.	Take-off strips	AC	1-1/2	Sand	6	7
Tappahannock Flight Strip	Tappahannock, Va.	Runway	Sand-asphalt	3	Selected sand and gravel	4-6	7
Turner	Albany, Ga.	Runways E-W, N-S, NE-SW, NW-SE, taxiways	AC	1-1/2	Clayey sand (SC)	6	10
Victorville	Victorville, Calif.	Runways 2, 3, 4 Taxiways Anchorage	AC None	2 -----	Silty sandy gravel	8	6-26 11.3 (avg)

General notes.

1. With the exception of the airfields listed below the quality of the S/C was not determined. In most cases the cement content was determ of either 50 or 80. In those cases where the CBR was determined it exceeded 80. (a) Houlton, S/C compressive strengths ranged from 6 60 psi.
2. CBR is either "in-place" or "undisturbed" value at time of field evaluation.
3. Coverages were computed from traffic data (intensity and distribution) in evaluation reports. In general the airfields were opened to tra condition. Cycles were converted to coverages using plate 1, Manual for the Evaluation of Flexible Pavements in the Zone of Interior (1 all cases. Coverage and performance data are to time of evaluation only.
4. Flexible pavement thickness required for given traffic was based on Corps of Engineers CBR Design Curves.

Subgrade Soil Type	CBR	Traffic Coverages	Traffic Wheel Load kip	Flexible Pavement Thickness Required for Given Traffic in.	Total Pavement Thickness used in.	Performance (To Time of Evaluation, 1944)
(GC, SC)	20	3,500	20	8	7-1/2	Local areas distressed.
Clayey sand (SC)	14	1,600	15	7-1/2	8 to 9-1/2	Built 1951, evaluated 1952. S/C nonplastic. No deformation. Usual S/C cracks.
Red micaceous clay (CH)	15	5,000	10	7	6 to 8-1/2	Chipped out on top to 2-in. depth. Badly cracked. Numerous low areas, probably due to displacement of base.
Sandy clay (SC)	35	-----	--	-----	6-8	Excellent condition.
Sandy clay (SC-CL)	20	4,000	15	7	8	Major pavement failures believed to be soil-cement; operations hazardous. Failed areas removed, filled with sand, surfaced with sand-asphalt. Subsequent performance good-very good.
Shell sand (Coquina)	45	-----	--	-----	9	Good condition. A few "birdbaths." Usual cracks in S/C.
Silty sand (SM)	35	260	15	3	7-1/2	Good condition.
Uniform fine sand (SP)	20	3,000	35	10-1/2	9-1/2 to 10-1/2	Cracks observed, not associated with overloading. Washboarding of pavement. Spot failures. 2-1/2 in. settlement over 50-ft length due to drainage line washout.
5-in. gravel Clayey silt (CL)	50 12	1,100	3	1-1/2	6-1/2	Good condition.
Sandy loam (CL, ML, SC)	17	1,100	30	10	9	Generally fair condition. Definite signs of overstress on S/C apron which has carried most of traffic.
Sand-clay, gravel-sand-clay. Poorly graded	28	-----	--	-----	6	Good condition.
Sandy clayey silt (ML)	15	6,000 (est)	30	12	7-1/2	Good condition.
Silty sand (SC)	35±	350	7	2	6	Unsurfaced area worn, rutted, broken, and cracked. 6-in. S/C base with 2-in. AC surface in good condition.
Plastic clay (CL, SC)	21	-----	--	-----	7-1/2	Very good condition.
Poorly graded fine sand and silt (SC)	10	1	15	<6	7-9	Very good condition.
Silty sand, clayey sand (SM, SC)	15	5,000	15	6-1/2	7-1/2	Evaluated 1952. Usual S/C cracks. No subgrade distress. Good condition. S/C nonplastic.
Clayey silty sand and gravel (GC, SC)	25	310	15	4	6-10	Surfaced S/C in good condition. Unsurfaced S/C showed excessive abrasion.

d prior to construction on the basis of wet-dry and freeze-thaw laboratory tests. In the evaluation reports the S/C bases were assigned CBR values
767 psi; (b) Long Beach, S/C flexural strength averaged 48 psi; (c) Morris, poor quality S/C; (d) Tappahannock, S/C modulus of rupture was

c in 1941 or 1942 and evaluated in 1944. That combination of coverages and wheel load was selected that would result in the most critical loading
ised Draft), prepared by the Waterways Experiment Station for the Office, Chief of Engineers, July 1954. Tire pressures were <100 psi in almost

too conservative in view of the severity of the tests in their present form.

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Proceedings of the American Society of Civil Engineers

PILE CONSTRUCTION IN PERMAFROST[a]

J. A. Pihlainen[1]

SYNOPSIS

General construction problems and practice in permafrost areas, notably Canada, are reviewed. Present design considerations for piles in permafrost involving the preservation of permafrost and the tangential adfreezing strength are summarized. Field operations with piles in permafrost areas including the types of piles used, site preparation, drilling or steam jetting pile locations, pile placing and the refreezing of piles are described.

INTRODUCTION

In the northern part of Canada, the climate is such that a portion of the ground remains frozen throughout the year. Some surface thawing does take place, depending on the locality and the insulating effect of the moss cover but, below this, the ground remains frozen throughout the year. These perennially frozen grounds are known as permafrost. The material in this perennially frozen state may include bedrock, gravel, sand, silt, clay, organic material, and ice. Thus permafrost is not the name of a new material but of the frozen equivalent of materials found in areas further south.

Permafrost is found in about one-fifth of the land area of the world. Almost one-half of the USSR, much of Alaska, and more than one-third of Canada are underlain by permafrost.(1,2) In Canada, almost all of the Northwest Territories, Yukon Territories, and the northern parts of British Columbia, Alberta, Saskatchewan, Manitoba, Ontario, Quebec and Labrador have permafrost (Fig. 1). The thickness of permafrost in Canada varies with location,

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a. Presented at the October 1958 ASCE Convention in New York, N. Y.

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being naturally deepest in the arctic and thinning out on the southern limits. At Resolute Bay on Cornwallis Island it is approximately 1300 feet thick,[3] at Norman Wells, N.W.T. it is about 150 feet thick,[4] while at Hay River, N.W.T. some patches of permafrost only 5 feet thick have been found[5] (Fig. 1).

Construction Problems in Permafrost Areas

The presence of permafrost or perennially frozen soil at a short distance below the ground surface presents unique difficulties to construction in northern Canada. The three principal properties of permafrost which give rise to these difficulties are its ice content, its thermal sensitivity, and its imperviousness to water.

The ice in a frozen soil acts like a cement, bonding the individual soil particles. The result is a material with considerable, often rock-like, strength. The ice can take the form of layers or lenses ranging from hairline size to 3 and 4 feet in thickness (Fig. 2), or it may be found as coatings over small soil particles as well as boulders. Some of the most spectacular ice deposits occur as chunks, wedges, or blocks buried in perennially frozen ground. The volume of ice in permafrost may be as much as six times that of the soil solids. Thawing of this ice can change perennially frozen soil from a firm bearing material to a slurry with no supporting power. However, all perennially frozen materials do not have these objectionable properties and some coarse-grained soils, as well as bedrock, are favorable foundation materials.

The thermal sensitivity of permafrost makes it a difficult material in which to carry out construction without causing some thawing of the frozen soil.

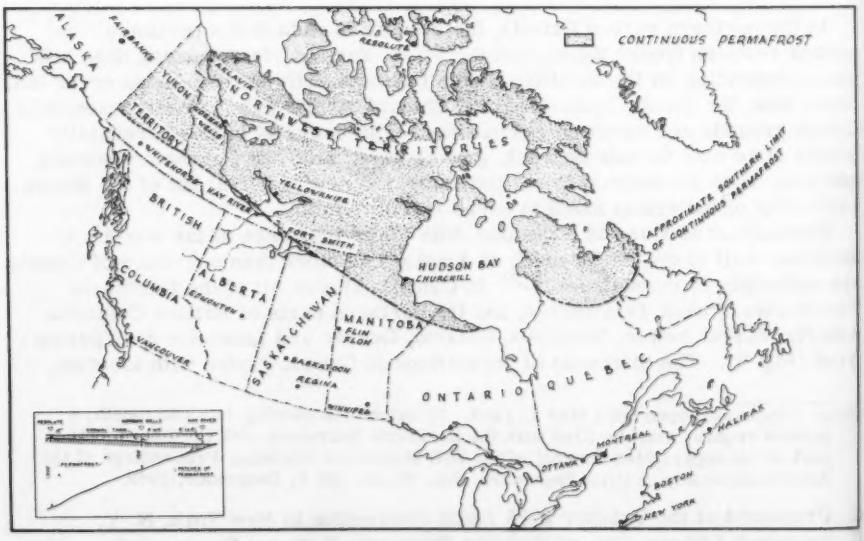


Fig. 1. The distribution of permafrost in Canada.

Permafrost conditions at an undisturbed site are in a delicate state of thermal equilibrium. So delicate is the thermal balance that the permafrost table is ever fluctuating with variations of climate and with the existing type of vegetation. The thermal sensitivity of permafrost is well illustrated by game trails which become slightly depressed paths as the permafrost thaws a little more under this compacted moss than under the surrounding terrain.

It is not strange that the introduction of a building to undisturbed terrain has such a pronounced effect on the permafrost table. Clearing of the trees, brush, and the moss cover start the change in thermal equilibrium. Steam jetting or excavation, and finally the heat losses from a structure complete the thermal disruption. Should the permafrost melt below the foundations of the building, then the building may slowly sink into the ground as the thaw continues. The problem can be described as trying to build a house on a block of ice.

The third, and often neglected, property of permafrost is its imperviousness to water. Rainfall and water from melting snow cannot drain into the ground because of permafrost and tend instead to form stagnant pools and lakes. The poor drainage which results from this fact complicates construction operations and imposes additional problems on the design of buildings in permafrost areas.

As long as water in the summer and snow in the winter provided necessary transportation routes, and buildings were of simple design, the slow and small thaw of permafrost was unimportant. Building foundations consisted mainly of mudsills or timber pads; roads were merely winter hauling trails; and airstrips, if any, were small cleared areas improved by some brush removal.

In recent years, however, much clearing of frozen ground has taken place for road and airfield construction and with the advent of heated basements for

Fig. 2. Split core of Aklavik perennially frozen sandy silt. The volume of ice is three times that of the soil. Darkened portions show ice lenses in one section of the core. (Photographic enlargement twice natural size.)

buildings, the peculiar properties of permafrost have made themselves very evident, especially to engineers. Portions of roads become unserviceable in a matter of months; airstrip construction in some locations has had to be abandoned; and building movements measured in feet have been recorded.

Construction Practice

Early construction practice treated permafrost as a "hard pan". Small buildings were supported by mudsills or pads on the ground surface, larger structures on piers which extended to or slightly into permafrost. Although the bearing capacity of the permafrost was sufficient to distribute the small building loads and the thaw of permafrost was slow and gradual, other faults in this general technique soon forced its abandonment. Frost action frequently moved supporting members up and down as much as 2 feet. In addition, heat loss from a structure would eventually lower the permafrost level and the foundations would settle. These destructive effects, sometimes occurring singly but mostly concurrently, would soon have the structure in distress (Fig. 3).

The difficulties of early construction clearly indicated the need for proper preliminary investigation before the selection and design of foundations in

Fig. 3. Crack in foundation wall ranging from 2 inches at top to hairline near ground level.

permafrost areas could safely be attempted. A thorough site survey, with its scope patterned on the importance of the structure, is a prerequisite to foundation selection.

If a site with favorable perennially frozen foundation material is not readily available, then the only recourse is the utilization of the perennially frozen ground which has poor engineering properties. For this type of design, every effort must be made to protect the existing permafrost. Foundation types which disturb natural ground conditions and which allow large heat losses from the structure to the ground are to be avoided. For satisfactory foundation performance, the problem is to estimate the probable thawing of permafrost for the life expectancy of the structure and to design the foundations for these conditions.

For large structures, present construction techniques favor the "embedment" of foundations in permafrost. Embedding the foundations in permafrost provides an anchorage or grip which resists frost action forces and allows for some thawing of the permafrost table without loss of bearing of the foundations. The excavation of perennially frozen soil, however, is costly because of its hard, almost rock-like quality and the fact that on melting most fine-grained soils with high ice contents turn into slurry. The embedment of foundation members by excavation is therefore a difficult and costly operation. An alternative method which provides for anchorage of the foundations in permafrost is to thaw a hole in the ground, and to drive the foundation member sufficiently into the thawed ground so that subsequent refreezing will establish the permafrost table at some distance above the bottom of the member. This is basically the present procedure used for pile foundations in Canadian permafrost areas.

Some Advantages and Disadvantages of Pile Foundations

The principal advantage of pile foundations is that they can be extended with a minimum of effort a sufficient distance into permafrost so that their function should not be impaired by thawing during the life span of the structure. In many cases such as poorly drained sites underlain by fine-grained soils with high ice contents, piles are the only type of foundation that can be used with any hope of success. When local timber is available, the high costs of transportation to the north can be saved. Thus the costs of timber pile foundations can be often justified for smaller structures such as dwellings. This is especially true for large projects where the cost of equipment required is spread over a number of installations.

An often-mentioned disadvantage of pile foundations is that basements are not usually possible with this type of foundation. In view of the problems with basement construction in the north,(5) the opinion of many actively engaged in northern construction is that such extra space as is provided by a basement can better be obtained by additions to the superstructure rather than by attempting any excavation under a northern building.

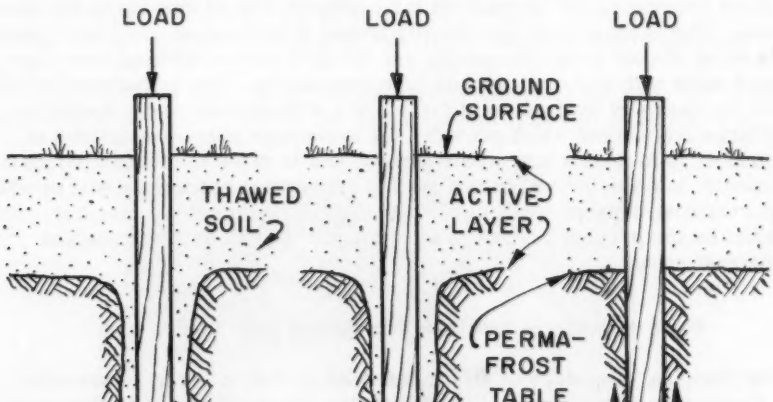
Design Considerations

Present pile construction practice in Canadian permafrost areas is to thaw, or drill holes at, the pile locations, to drive or place the piles, and then allow the piles to refreeze. When a pile is first placed, it transfers its load to

permafrost by end bearing. As refreezing begins, the load is distributed partly by the bond between the pile and the frozen soil (tangential adfreezing strength) and partly by end bearing. When the pile is completely frozen the load is thought to be transferred wholly by tangential adfreezing strength (Fig. 4). The tangential adfreezing strength is used not only to distribute the load to permafrost but also to combat frost heaving forces. Accordingly, the design problem is two-fold—first to predict the thawing effect that the structure will have on permafrost during the life of the building, and second to determine what tangential adfreezing strength can be safely assumed for distributing the load to permafrost and combating frost heaving forces.

Preserving Permafrost

The effect of heat losses from a structure during its life span which tend to increase gradually the depth of thaw (and thus reduce the tangential



① IMMEDIATELY AFTER DRIVING, LOAD TRANSFERRED BY POINT BEARING.

② DURING FREEZING, LOAD TRANSFERRED BY POINT BEARING AND ADFREEZING STRENGTH.

③ COMPLETELY FROZEN, LOAD TRANSFERRED WHOLLY BY ADFREEZING STRENGTH.

Fig. 4. Load bearing cycle of pile after steam jetting.

adfreezing strength) is difficult to predict mathematically. Some calculations on the depth of thaw in frozen ground under various surfaces have been made.(6) The correlation of such calculations with the effect of pile supported structures having an open air space between the ground surface and the floor is still required.

It is usual to minimize heat losses to the ground or to preserve permafrost conditions as much as possible. This preservation of permafrost is assisted by keeping the disturbance of the natural vegetative cover to a minimum, by the provision of an air space between the floor of the building and the ground surface through which cold air may circulate during the heating season, and by providing ground insulation in the form of additional moss or dry organic material under the building.

Adfreezing Strength

In anchoring a pile in permafrost, the grip of the frozen soil on the pile is utilized. This grip or tangential adfreezing strength may be defined as the resistance to the force that is required to shear off an object which is frozen to the ground and to overcome the friction along the plane of its contact with the ground.

Russian investigations(7,8,9) have shown that the tangential adfreezing strength depends on:

(i) the texture and density of the soil
(ii) the temperature of the soil
(iii) the moisture content of the soil
(iv) the area of contact, and
(v) the nature of the contact face, i.e. smooth or rough.

Some of these variables are illustrated in Figs. 5, 6, 7 and 8. In general the tangential adfreezing strength:

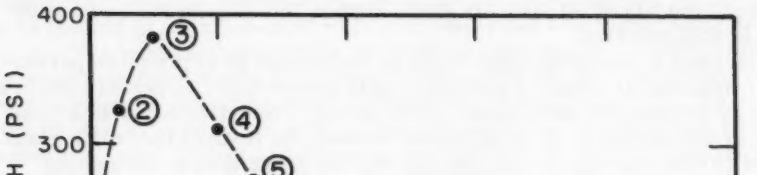
(i) increases as soil density increases, and
(ii) increases as the degree of saturation increases.

Nees(10) conducted model tests on a sandy-silt with 1-1/4-inch standard steel pipe and found that the general trends appeared to be the same but that the model tests yielded much lower values.

Although some information on the values of tangential adfreezing strengths is available, virtually no records are available on frost-heaving forces. This has led to the extensive use of a "rule of thumb" for embedment of piles in permafrost, e.g. embedment or anchorage of piles in permafrost should be equal to at least twice the depth of seasonal freezing and thawing during the life of the building.(1)

Types of Piles

Wooden piles are preferred because of their cost advantage in areas where they can be procured locally; relative conductivities of wood, concrete and steel also favor the use of wooden piles in permafrost. Some precast concrete piles and steel pipe piles have been used but higher costs have curtailed their extensive use. Cast-in-place concrete piles are not used in permafrost areas. The following estimated costs of a 20-ft pile delivered to Inuvik illustrate well the reason for preferring local timber piles.(11)



LEGEND:

No.	TYPE OF SOIL	TEMP (°F)	COEF OF SAT (%)
1	CLAY	26·5	77
2	SAND, FINE	26·5	76
3	SAND, MEDIUM	26·5	78
4	SAND, COARSE TO FINE	26·5	79
5	SAND, COARSE	26·5	97
6	GRAVEL, FINE	26·5	77
7	GRAVEL	26·5	79

(AFTER SUMGIN REF 9)

Fig. 5. Tangential adfreezing strength between wood and various soil textures.

20-ft Pile	Cost
local spruce with preservative	$ 20.00
precast concrete (cast at site)	80.00
creosoted, from Edmonton	100.00

Wooden Piles

Local spruce timbers are most commonly used for piles in the treed permafrost areas. Timber sizes will vary with locality but 20-ft piles with minimum 8-in. tips should be readily available. In the development of the new townsite of Inuvik at the mouth of the Mackenzie River, N.W.T., which is to replace Aklavik, local 20-ft spruce piles were used but in addition spruce piles 40 ft long with 8-in. tips were obtained for special requirements from the Liard River region.

Pile preparation may or may not include the stripping of bark. If a preservative is to be used, stripping of bark and preservative treatment is necessary for that portion of pile which will be in thawed soil. It is very difficult to get a preservative to penetrate spruce by any treatment method, including pressure treatments, except by a diffusion process. With this process the preservative is usually applied to green timber by brush in the form of a slurry. The piles are then stacked closely, covered with waterproof paper to prevent rapid drying, and allowed to stand until the preservative has diffused into the wood. The cost of a 20-ft spruce pile in place at Inuvik, including the cost of the pile, diffusion treatment, handling, steaming and placing the pile, is estimated to be $30 per pile.(12)

Concrete Piles

The use of site precast concrete piles has been limited in permafrost areas because of the high costs of transportation and the need for skilled workmen.

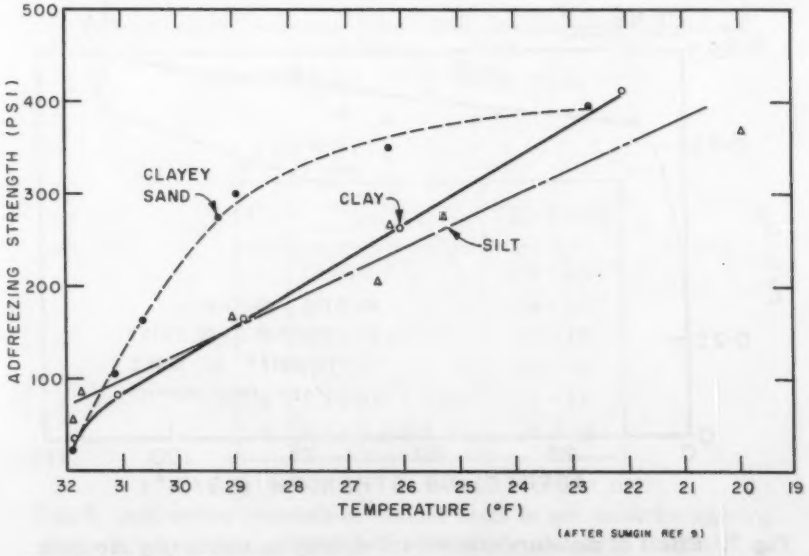


Fig. 6. Effect of temperature on the tangential adfreezing strength for various soil textures.

Cast-in-place concrete piles are not used in permafrost areas because of the problems of curing concrete which is surrounded by permafrost.

Steel Piles

As with concrete piles, steel piles are not in extensive use mainly because of their high cost. They have, however, been used in some communities where steel pipe was available as scrap or surplus, notably Norman Wells, N.W.T. They are easily handled, may be quickly lengthened, cut or capped by welding, and are slightly flexible as to position after driving.(4)

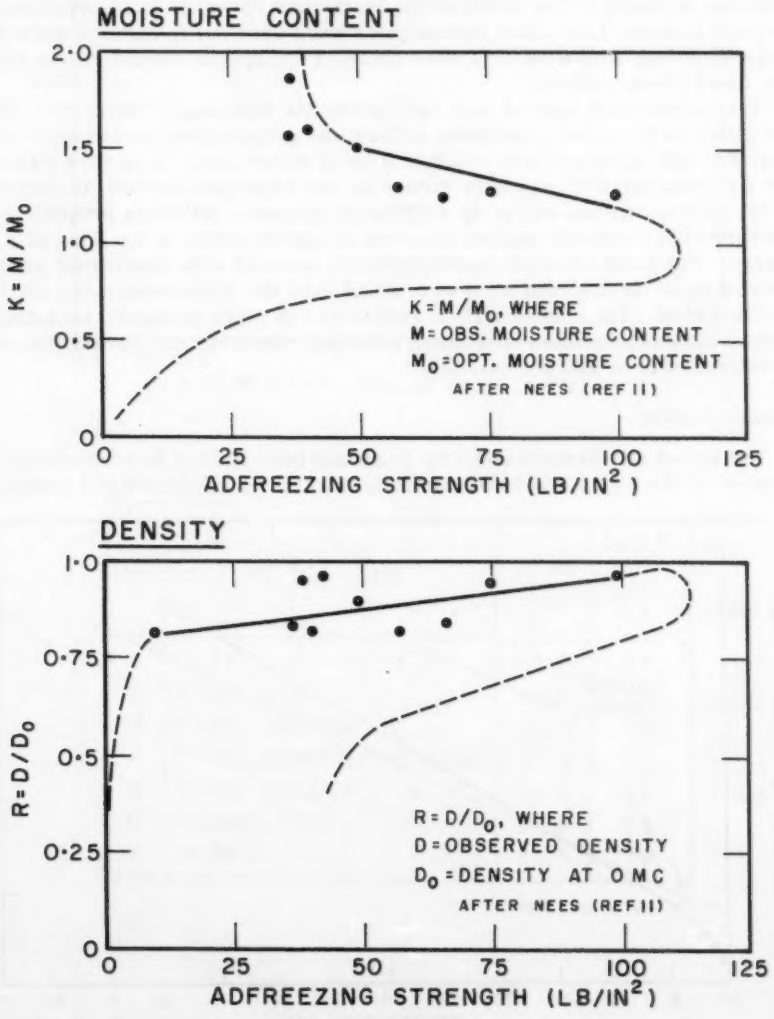
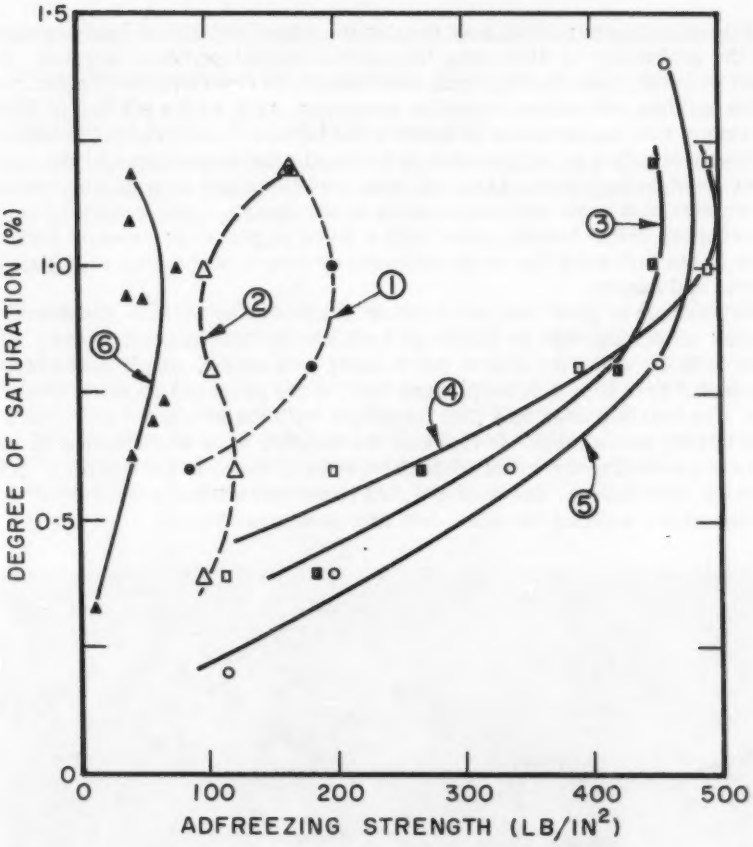


Fig. 7. Effect of moisture content and density on adfreezing strength.



LEGEND:

No.	SOIL	TEMP (°F)
1	CLAYEY	29-30
2	SILTY	29-30
3	CLAYEY	11-15
4	SILTY	11-15
5	SANDY LOAM	11-15
6	SAND-SILT	18-25

(AFTER NEES REF 11)

Fig. 8. Adfreezing strength of various soils to wet wood for varying degrees of saturation.

Site Preparation

Pile-placing operations may involve the use of relatively heavy equipment and the probability of disturbing the natural vegetative cover is great. If it is at all possible, pile-placing equipment should be restricted to the perimeter of the building site where remedial measures, such as the placing of additional organic fill, can be taken to restore the terrain to its original condition.

Frequently this is not possible and some initial disturbance of the organic cover must be accepted. At times, site conditions are so poor for equipment movements that some site preparation is necessary. This generally requires covering the entire building area with a layer of gravel or crushed rock. In some cases this work has been neglected or overlooked with a resulting loss in time and money.

An example of good site preparation is afforded by the pile construction carried out during 1950 by the Giant Yellowknife Gold Mines Co. Ltd., Yellowknife, N.W.T. After the site of the building was chosen, small boxes (approximately 2 ft by 2 ft by 1 ft deep) were built at the proposed location of each pile. The building area was then backfilled with "mine-muck" fill (waste rock from mining operations). In this way the building area was levelled in addition to protecting the moss from the tracks of the crawler tractors. After the piles were driven, the wooden guard boxes were found to be very convenient for wedging the piles into true positions (Fig. 9).

Fig. 9. Close-up of driven pile, cut off to desired elevation and wedged into line, at the Giant Yellowknife Gold Mines, Yellowknife, N.W.T. Tarpaper collar used to reduce evaporation of diffusion type of preservative.

Placing Piles in Permafrost

At present two methods of placing piles in permafrost are used in Canada:

a. Steam jetting (thawing) the permafrost at the pile location and driving the pile, and
b. Drilling and placing the pile in the core hole.

Both methods should be reviewed before a final foundation design is chosen since each method has special merits when considered in relation to final costs and permafrost conditions.

Steam Jetting

Steaming pile locations in permafrost areas is accomplished by advancing a steam jet into frozen ground under its own weight or with help from the person guiding the jet. When the steam jet has reached the desired pile depth it is withdrawn from the hole and the pile is placed in the thawed location. A pile driver then drives the pile to refusal, i.e. to frozen ground.

As would be expected, the mode and efficiency of steam jetting depends upon many variables: the type of steaming equipment that is readily available, the steaming interval, the type of soil, the moisture or ice content of the soil,

Fig. 10. Steam jet used at Giant Yellowknife Gold Mines, Yellowknife, N.W.T. The pipe is 1 inch in diameter with 1/4-inch nozzle.

and the temperature of the frozen soil. The steaming characteristics of a locality are best determined by field trials.

A steam boiler from 25 to 50 BHP, preferably 50 BHP, is required. Steam pressures from 50 to 120 psi have been used; the higher pressures appear to give better results. At Inuvik in 1958, a 50 BHP oil-fired boiler supplied five 3/4-inch diameter steam jets at 100 psi and used 5 barrels of water per 10-hour day and approximately 12 gallons per hour of fuel oil. Steam jet piping is generally 3/4-in. to 1-1/4-in. pipe, open or slightly crimped to give a better jetting action (Fig. 10). Hemstock[4] has noted that, except in large gravel, special bits such as a chisel bit did not speed jetting. For relatively long piles (longer than 20 ft), a long steam jetting pipe is difficult to handle. In these cases, steam jetting a pile location is started with a short jetting pipe, say 8 ft long, and completed with the longer jetting pipe.

Soils with some ice steam most efficiently for pile driving, i.e. they produce the desired diameter thaw in the shortest time. The rate of thaw in "dry" soils can sometimes be increased by the addition of some water to the thawed hole. Steaming penetration through thick ice is fast but only a small hole slightly larger than the jet is produced. An experienced operator will soon recognize ice formations and will slow down the jet travel in it. Ice lenses in soil, ranging from 1/2 in. to 3 in. in thickness, frequently form ice ledges which reduce the effective diameter of the thawed hole. Working the steam jet up and down beside the pile as it is driven remove these ledges if they impede driving.

One of the hardest materials to thaw by steam jet is frozen organic material. The extremely high ice content solidly bonds the organic particles and fibres. Steaming produces a small hole slightly larger in diameter than the jet and the resulting mat of thawed organic material impedes further thawing. Frequently woody particles such as tree trunks or branches completely stop jet penetration. Several closely spaced holes must then be thawed through the organic material before the desired diameter of thawed hole is produced. If extensive organic deposits occur, the use of jets which direct the steam radially from the pipe may increase steaming efficiency.

The steaming penetration rates for various projects at Norman Wells, Aklavik, and Inuvik have been summarized in Table 1. The rates at Inuvik and Aklavik vary from 3 to 24 ft per hour of steaming with differences attributed mainly to the occurrence of organic material, or boulders as at Inuvik. It will be noted that the steaming penetration rate for Norman Wells of 50 ft per hour of steaming is double the fastest rate for the Aklavik region. The Norman Wells rate is based on trouble-free steaming, when boulders or organic material were not encountered. These steaming penetration rates merely indicate the relative magnitude of this effect so that some first appraisal of the steaming time can be made.

An interesting steam jetting technique was developed at Inuvik, during the summer of 1956. Stones and boulders in the frozen gravel sometimes hindered steam jetting but were most troublesome during pile driving. Stones would frequently be loosened by pile driving and act as wedges deflecting the piles out of line by as much as 3 ft. At times, stones would be forced to the bottom of the hole and the pile could not be driven to full depth.

The steaming technique developed by field trials was to steam the hole to the required depth and diameter with the least possible delay (considering the type of soil). This steaming penetration rate (steam pressure 100 psi) varied from 10 to 94 ft per hour but averaged 40 ft per hour for 70 pile observations.

The steam jet was then left at the bottom of the hole to "bell out" a thawed area into which stones and boulders could be pushed by the pile as it was driven. Various "belling out" times were tried. The average time for 70 successful locations was found to be 18 minutes. Steaming penetration rates including the time for "belling out" varied from 8 to 39 ft per hour for 70 pile locations and averaged 21 ft per hour. The "belling out" steam jetting technique is illustrated in Fig. 11.

Although the term "pile driving" is used in permafrost areas, piles are not driven directly into permafrost. After steaming pile locations, the piles are placed in thawed soil, at times a hole filled with a soil slurry, and pushed, tapped or driven to the depth of steaming. The type of pile placing equipment used varies with availability. At Norman Wells, for light driving to a depth of 16 ft, Hemstock[4] found that a light, drop-hammer mounted on a small crawler-type tractor was most satisfactory. For a small number of piles, a gin pole, fitted with a block and tackle, and a hand-operated winch has been used. At Giant Yellowknife Gold Mines, piles were "pushed" into the thawed soil with the winch and cable of a D-8 caterpillar tractor. Drive weights used have varied from 800 to 2500 lb.

As the piles are forced into the thawed soil, the soil slurry usually oozes out at the top since the surrounding permafrost is impervious. At times, especially if the holes have been oversteamed, the piles "pop-up" or float in the

TABLE 1

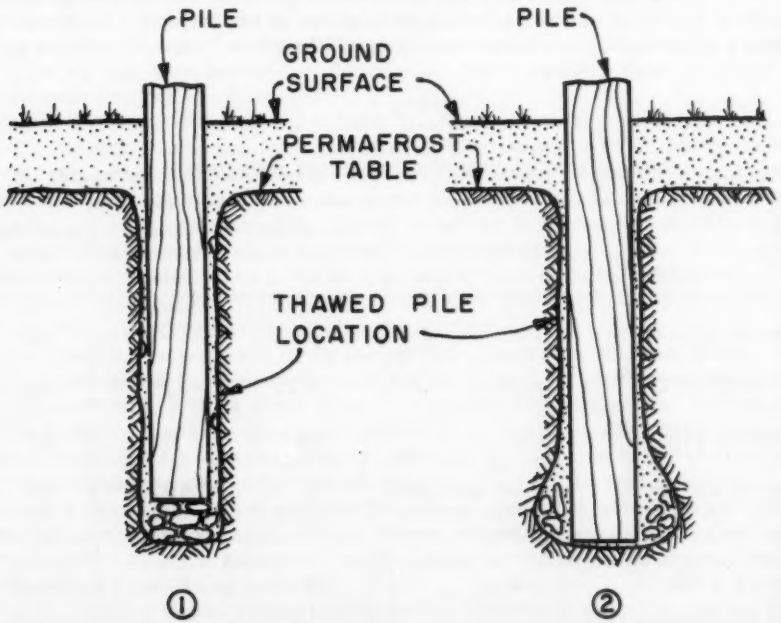
SOME STEAMING PENETRATION RATES AND DETAILS

Location	Predominant Soil	Depth of Steamed Hole (ft)	Steam Pressure (psi)	Steaming Rate (ft/hr)
Norman Wells 1947 [5]	silty clay	16	80-100	50
Aklavik 1952 [19]	silt	18-21	75	24
Aklavik 1953 [20]	silt	20	90	15
Inuvik 1955	fine-grained	20	100	20
	sandy gravel	20	100	10
	organic	20	100	10
Inuvik 1958	gravel	20	100	3-10

soil slurry. This necessitates anchoring or wedging the pile in place and in line until it is sufficiently frozen in.

At Inuvik, pile alignment during driving is difficult because of stones which deflect the piles and the straightening of a considerable number of piles is necessary. Average realignment is about 4 inches although some piles have been straightened as much as 12 inches. The procedure used at Inuvik is to delay re-alignment for about one month after pile driving since wedging is reported to be ineffective immediately after driving. One or two holes, depending on the amount of straightening, are steamed on the side to be moved. For alignments of approximately 6 inches or more, steaming is carried out to the bottom of the pile although usually the steaming depth is approximately two-thirds of the pile length in the ground. The piles are pulled into alignment with a winch and long wooden wedges are inserted to hold the pile in position. Less than 1 per cent of the piles have broken in straightening operations at Inuvik.

The refreezing of piles in permafrost depends upon many variables of which the steaming interval is probably the most important. In general, individual piles or small groups of piles with moderate steaming refreeze



① STONES DEFLECT PILE AND PREVENT DRIVING TO FULL DEPTH.

② STEAM JET LEFT AT HOLE BOTTOM APPROXIMATELY 15 MIN. TO THAW BELL-SHAPED REGION TO WHICH STONES ARE PUSHED BY PILE.

Fig. 11. "Belling out" technique developed at Inuvik for steaming in stony gravels.

sufficiently for construction in a matter of weeks in continuous permafrost. However, large pile areas, especially where steaming is difficult and when re-steaming for pile alignment is necessary, or even smaller groups of piles in areas of sporadic permafrost, may require as much as a year to refreeze before the risk of heaving is over.

The following observation on pile refreezing at Aklavik illustrate the refreezing phenomenon. It should be noted that the results presented here are relevant only to the Aklavik region. In general, longer refreezing intervals can be expected in more southerly locations; shorter intervals for more northerly locations.

The refreezing of four piles at Aklavik was observed during 1953. The piles were each steamed for approximately 1-1/2 hr for a 20-ft depth using steam at 90 psi. Thermocouples were attached to the piles to record temperatures at 2-, 8-, 14- and 20-ft depths. Observations began one or two days after the piles were driven (early July) and continued every second or third day until the middle of September. Table 2 records the refreezing intervals for the various depths.

One of the findings of these observations was the fact that the refreezing of piles in the Aklavik area proceeds in a matter of weeks for moderate steaming. The importance of careful steaming is also emphasized by the results shown in Table 2. Thus at the 20-ft depth, refreezing can occur as late as 39 days after driving, due to oversteaming, although one day or less is the general case. Similarly the range of refreezing times for the 14-ft depth varied from 2 to 28 days. For the 8-ft depth, the time of refreezing was 44 or more days.

Similar refreezing records were collected at Inuvik, during 1955. These results indicated that moderately steamed pile locations were refrozen up to a depth of 11 ft one month after steaming. This is in agreement with the Aklavik refreezing records obtained during 1953. However, the longer steaming intervals observed at Inuvik during 1958 have suggested that at least one winter interval of refreezing should be allowed before construction begins on the piles.

Fig. 12. Piling operations at Inuvik during August 1958. Note gravel pad over building site. Three steam jets are steaming pile locations at building site on right and the pile driver is completing driving at building site on the left.

Drilling Holes for Piles

The main advantages of drilling holes for piles are that disturbance of the perennially frozen condition is kept to a minimum and a reasonably accurate record of the soils and permafrost conditions at each pile location is obtained. These advantages are of particular value for construction in areas where permafrost occurs in "patches" of limited areal extent (commonly referred to as "sporadic permafrost areas"). In these sporadic permafrost areas, the temperature of permafrost is close to the thawing temperature and in some cases, if once thawed, will not refreeze. It may also be uneconomical to destroy the perennially frozen condition and accordingly the future existence of permafrost at each pile location is essential. Drilling the pile locations assures a minimum of disturbance to permafrost. In addition it offers a convenient opportunity to install piping to provide for future refrigeration of pile locations in the event of unanticipated thawing of the foundations.(14,15) The provision of refrigeration piping in steamed and driven piles is difficult since the piping must be protected from damage during driving. In areas of "continuous" permafrost (where permafrost is found everywhere under the natural surface, is relatively thick and has a temperature considerably below thawing) the advantages of drilling are not paramount except when quick refreezing is essential or when the project is large and drilling allows a faster rate of pile installation.

It is probably because of these reasons that drilling pile locations has not been common in the Canadian North. It has been utilized at Inuvik in 1957 for the pile foundations of a powerhouse and large heated oil storage tanks. The large number of piles involved, the close spacing of the piles and the importance of the structures suggested that extreme care was needed in keeping the disturbance of permafrost to a minimum. Drilling, rather than steaming, the pile locations was therefore started during September 1957. Two truck-mounted seismic shot-hole drill rigs drilling 24- and 18-inch diameter holes experienced considerable difficulty with stones and boulders. The procedure developed, although not completely satisfactory, consisted of drilling the frozen stony material with a 20-inch diameter straight blade auger using

TABLE 2

PILE REFREEZING INTERVALS, AKLAVIK 1953

Depth (ft)	Time of Pile Refreezing			
	Pile-1	Pile-2	Pile-3	Pile-4
2	-	-	-	-
8	44 days	> 55 days	> 39 days	> 39 days
14	7 days	28 days	2 days	20 days
20	< 1 day	< 1 day	39 days	1 day

steam to thaw and loosen the stones when drilling became difficult. A commercially available special carbide insert auger performed well in frozen silt and ice although stones did chip and break away the carbide teeth. Drilling a hole to a depth of 26 feet took approximately 1-1/2 to 2 hours.

At Frobisher Bay,(16) in 1958, 24-foot deep holes for piles were drilled with an 18-inch diameter auger but as at East Three, pockets of boulders up to 10-inches diameter caused much difficulty. The drilling procedure at Frobisher Bay consisted of drilling a 2-inch diameter hole at each pile location and steaming the hole prior to augering. However, the boulders caused extensive damage to the auger carbide inserts. Frost cutter and core bits were also used with little or no success.

Explosives were then tried to loosen the boulders. After blasting, the auger was able to excavate the pile location with much more success. The method consisted of drilling three 2-inch diameter holes equally spaced 20 inches from a centre hole at each pile location. The centre hole was left open and the outside holes were loaded with 1-1/8 x 8 inch cartridges of 75 per cent Forcite at 12 inch intervals. These holes were bottom primed with short period caps to give a maximum delay between each hole in the location. Since a collar of 5 feet was maintained on each loaded hole, no surface cracks or caving was apparent.

In contrast to Canadian practice, pile placing in the permafrost areas of Alaska has been carried almost entirely by drilling methods. A typical example of such an operation was during the winter of 1956 when Morrison-Knudsen Inc. drilled 18-inch diameter holes for 30-foot wood piles at Bethel, Alaska.(17) After several field drilling trials they decided to use a special auger equipped with a special drilling head. This tool has four cutting surfaces with square carbide inserts that can be reversed to utilize the other sides. Then the head can be shifted into an upside-down position to provide more cutting surfaces. As frozen silt is highly abrasive, drilling into this frozen soil can cause excessive wear on any cutting heads. After drilling, piles with refrigeration piping attached were lowered into the holes. A sandy-silt slurry was pumped around the piles as backfill and refrigeration equipment froze in the piling.

CONCLUSION

Pile foundations are a potentially excellent foundation type in permafrost areas. Capable of carrying heavier than ordinary building loads, not immediately affected by the gradual thawing of permafrost or by frost action, relatively simple to install, and utilizing local materials, an increase in their present use can be expected. Design considerations of piles in permafrost consists of predicting the effect that the structure will have on permafrost during the life of the building and then to determine what tangential adfreezing strength can be safely assumed for distributing the load to permafrost and combating frost heaving forces. The field techniques of pile construction in permafrost involve many variables, some best determined by field trials. It is hoped that the methods described will help engineers in a first appraisal of the use of pile foundations in permafrost areas.

ACKNOWLEDGMENTS

Much of the information reported has been collected at many construction sites in the Canadian North. Individual acknowledgments for assistance are not possible, and appreciation to construction personnel in the Canadian North must be recorded collectively.

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CONSTRUCTION AND MAINTENANCE OF UNDERSEEPAGE CONTROL MEASURES

W. J. Turnbull,[1] F. ASCE and C. I. Mansur,[2] F. ASCE

This is one of three companion papers on underseepage and is referred to as companion paper No. 3. The first paper is entitled "Investigation of Underseepage—Mississippi River Levees" and is referred to as companion paper No. 1. The second paper is entitled "Design of Underseepage Control Measures for Dams and Levees" and is referred to as companion paper No. 2.

SYNOPSIS

A summary of basic principles involved in the construction and maintenance of various underseepage control measures is presented in this paper, together with an appendix describing a field study of a system of relief wells installed along a reach of levee along the Mississippi River.

Underseepage Control Measures

The following paragraphs describe various methods which may be used to control or partially control underseepage. Basic and pertinent principles involved in construction and maintenance of the various methods are set forth.

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Relief Wells

Relief wells can be installed in a hole made by either the reverse rotary method, the casing method, or other method that will not remove excess material from the foundation. The reverse rotary method of drilling well holes in sand is basically drilling by suction since material is removed by suction pipe. The walls of the hole are supported by seepage forces acting against a thin film of fine-grained soil on the walls, created by maintaining a head of water in the hole several feet above the ground-water table. A temporary casing may be used to support the walls of a hole during drilling and placing of well screen, riser pipe, and gravel filter. It may be set by any approved method that will not create a cavity around the outside of the casing.

Samples of the foundation soil should be obtained at 2-ft intervals during the drilling of each well. When the reverse rotary method of drilling is used, the samples can be obtained by catching and decanting samples of the effluent from the drill rig in a large bucket or other suitable container. When the wells are drilled by the casing method, the samples can be obtained from the

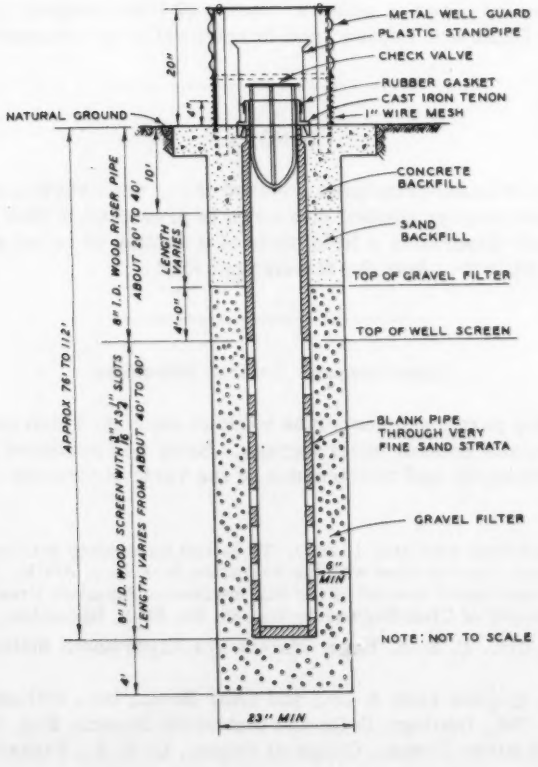


Fig. 1. Relief well and appurtenances

bailer or hoisting bucket being used. The purpose of this sampling is to determine the depth at which the screen section of the well can be started and also to determine the existence of strata of silt, silty sands, or very fine sands through which unslotted (blank) sections of pipe should be used rather than slotted screen.

A typical relief well is illustrated in Fig. 1. The riser pipe and screen should be constructed of machine-banded wood staves of either untreated redwood, or treated Douglas fir, white pine, or southern yellow pine. The staves should be at least 1-1/8-in. thick and milled from clear, seasoned lumber. Douglas fir, white and southern yellow pine should be pressure-treated with creosote or creosote-coal tar solution with a minimum retention of 8 lb per cu ft for Douglas fir and 10 lb per cu ft for white and yellow pine. The pipe should be banded with heavily galvanized, 6-gage winding wire with 3-in. spacing between wire turns. Wooden plugs are usually used for closing the bottom of the well screens. Bituminous coated corrugated metal pipe may also be used for well screens and riser pipe. The screen portion should be coated after it is perforated. However, wood pipe is considered preferable for relief wells.

The well screen and riser pipe should be assembled, or partly assembled, and the bottom of the well screen plugged before the hole for the well is completed. Each joint of pipe and the plug in the bottom of the screen should be fastened securely. Guides should be attached to the assembled riser pipe and screen which will center the assembly in the well and hold it while the filter gravel is placed. The guides must permit extension of the tremie to the bottom of the hole for the well. A small allowance should be made in setting the top of the well pipe to take care of settlement that may occur during surging and pumping, in order that the top of the well pipe will be at the design elevation.

No filter gravel should be placed before the well screen and riser are installed, as the amount required below the screen cannot be predetermined nor placed accurately enough to insure that the top of the riser pipe will be at the correct elevation. Filter gravel should be placed only after the screen and riser pipe have been lowered to the correct depth and the top of the riser pipe has been set slightly above the design elevation. The filter gravel should be placed by lowering the tremie (with narrow slots or small perforations) to the bottom of the hole and then filling it with filter gravel. (The hole for each well should be overdrilled by at least 4 ft to provide space at the bottom for filter gravel which may become segregated when the tremie pipe is first filled.) The tremie should be raised slowly to allow the gravel to run out of the bottom as additional filter material is fed in at the top. The tremie pipe must be kept filled with filter gravel above the water surface and must be handled at all times in a manner that will prevent segregation of the filter as it is placed. Filter gravel should be placed to an elevation at least 5 ft above the top of well screen to allow for settlement during development of the well.

If a temporary casing is used, the filter gravel should be placed in increments not to exceed 2 ft. The tremie and casing should be raised in increments approximately equal to increments of filter gravel placed, except that at no time before completion of the operation should the bottom of the casing be less than 1 ft below the top of the filter. Alternate placing of filter and withdrawing of casing should be continued until the filter has been placed to the desired elevation.

Material for the filter around the riser and screen should be a washed sand gravel free from any adherent coating, any vegetable matter, or elongated particles in quantities considered detrimental. In general, the filter should meet the gradation requirements shown in Fig. 2, and it should not be skip-graded.

A well should be properly developed after installation by surging and pumping to remove the muddy water used in advancing the hole and to develop the filter. One method for surging is to raise and lower a block made of a heavy rubber disk between two steel disks mounted on a steel rod. The rubber disk should have a diameter approximately 1 in. less than the inside diameter of the well and the steel disks should be 1 in. in diameter smaller than the rubber disk. In some cases it may be desirable to mount two disks approximately 5 ft apart on the steel rod. The surge block should be raised and lowered at a rate of approximately 5 ft per sec. The top of the riser pipe is usually set about 4 in. above the top of the concrete backfill around the well. The ground immediately around the well should be left flush with the top of the concrete and should be graded to drain to any adjacent lower ground. Although many relief wells have been installed with only the tenon end of the wood riser projecting above the concrete backfill, a more permanent top for a well is a cast iron fitting (Fig. 1).

After a well has been developed it should be subjected to a pumping test during which the rate of flow, drawdown, and the rate of sand infiltration into the well should be carefully determined. If practicable, the pumping test

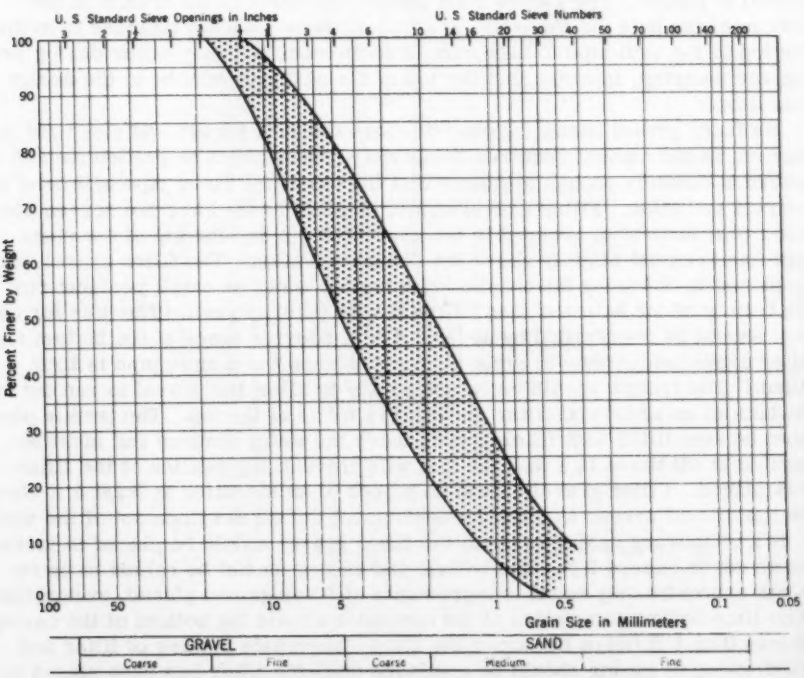


Fig. 2. Gradation of filter gravel for well screen with 3/16-in. slots and alluvial sands in the Mississippi River Valley

should be made when the water table is at or above the top of the well screen to insure flushing and to check on the stability of the entire length of screen and filter.

When it is desired to check the inflow of sand during a pumping test, the flow from the well should be discharged into a suitably baffled stilling tank (Fig. 3) to cause any fine sand in the water to settle out. The inflow of sand into a well during a pumping test can be determined in the following manner:

a. Determine the depth to top of sand in the well to the nearest 0.01 ft with a flat-bottomed sounding device and steel tape before starting the pumping test.
b. At 15-min intervals during pumping, determine the exact depth (nearest 0.01 ft) to sand in the well and amount of sand collected in stilling tank (to nearest 0.1 pt).
c. Total inflow of sand into the well should be computed for each time interval as the test progresses.

In the event sand or other material collects in the well as a result of the pumping test, it should be removed.

Installation and pumping test data should be recorded on a suitable form.(1) Any unusual conditions encountered should also be recorded.

The tops of relief wells should be protected with a suitable guard (e.g., Fig. 4), and backflow of surface water into the well should be prevented by an effective rubber gasket and check valve (e.g., Fig. 5).

Relief well flow is somewhat greater than natural seepage during relatively low flood stages. This increase in seepage can be minimized by providing each well with a standpipe (Fig. 6) to raise its discharge elevation 1 to 3 ft above natural ground surface. As soon as artesian pressure develops to such an extent that water begins to spill over the top of the standpipe, it should be removed and the well system allowed to operate as originally designed.

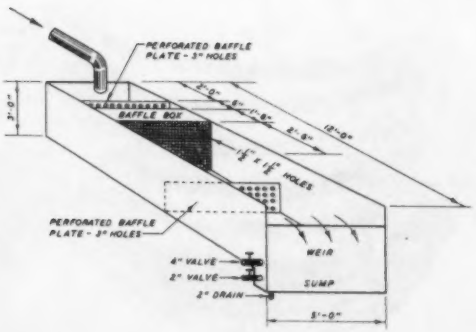


Fig. 3. Sand collection tank for checking stability of wells.

Relief wells adjacent to ditches or intakes to drainage structures or pumping stations along the levee should be provided with a check valve, standpipe for access, and a horizontal discharge pipe (Fig. 7). Relief wells installed in areas subject to flooding by sanitary or industrial sewage should be provided with the type of rubber gasket and check valve shown in Fig. 5, an outfall pipe equipped with an automatic flap gate, and a well top of the type shown in Fig. 8. These flap gates should be of cast iron with bronze facing. They should be double-hinged and bronze-mounted, and should close with a slight vertical angle. Bolts and washers should also be of bronze or brass.

Relief wells require a certain amount of nominal maintenance to insure their continued and proper functioning. In order for relief wells to function properly, it is essential that they be kept free of sand, silt, organic matter, or any other material that would retard free flow. Relief wells should be inspected once a year, preferably immediately prior to normal high-water seasons, and more often during major high waters for the purpose of detecting vandalism, abuse by thoughtlessness or carelessness, unauthorized use of the wells or piezometers, or other irregularities. Each well should be sounded every two or three years and after each major high water to see that it is free of trash and any obstruction, and to determine the amount of sand, silt, or other material that may have settled at the bottom of the well. Any trash, obstruction, or sediment to a depth greater than 1.5 ft in the well should be removed.

All wells requiring removal of sediment should be pump-tested after cleaning to see if any appreciable loss of efficiency has resulted from foreign

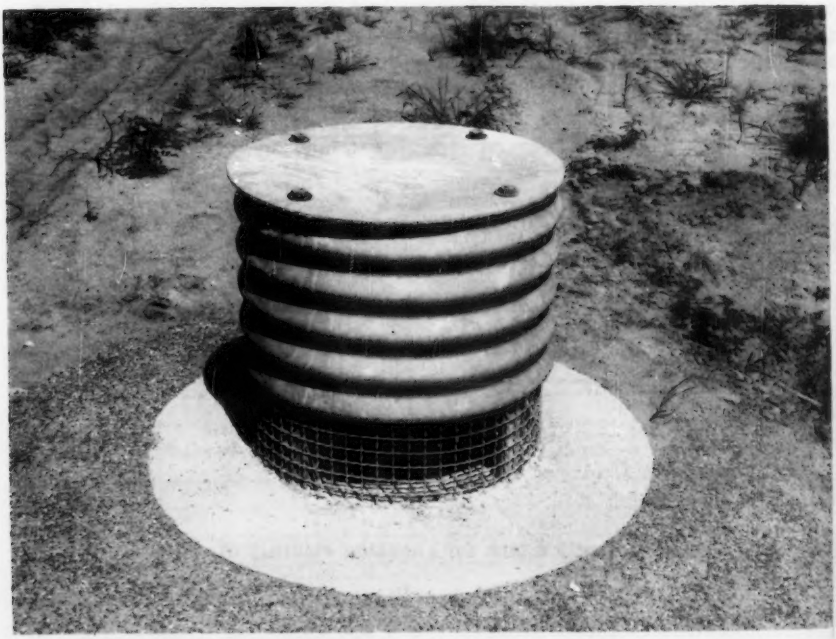


Fig. 4. Relief well with metal well guard

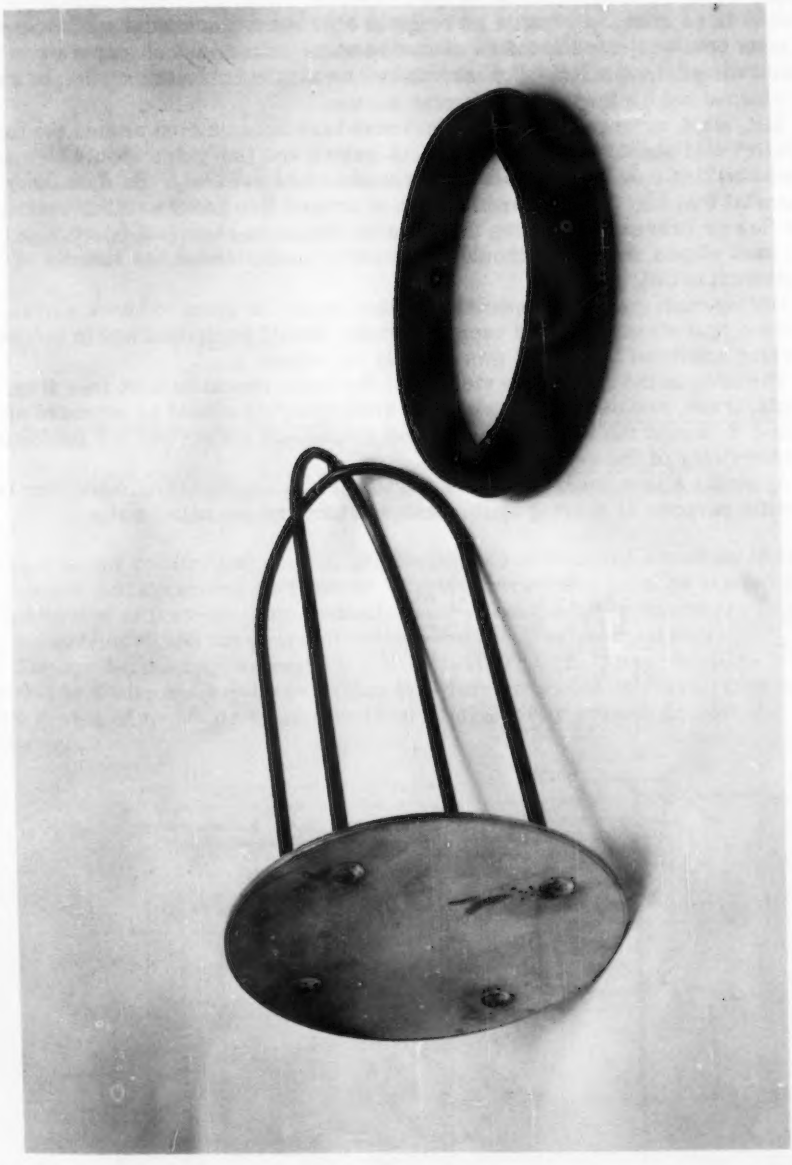


Fig. 5a. Check valve and gasket

material entering the well. In addition, all wells should be pump-tested at least every 5 to 8 years. If the pumping test indicates that the specific yield is 20% or more lower than the original specific yield, the well should be surged in an effort to restore its original efficiency. Individual wells known to have been subjected to inflow of muddy water as a result of inoperative check valves or removal of check valves or vandalism should be pumped and/or cleaned before the next high-water season.

Silt, sand, or organic material that may have accumulated around the top of a relief well should be removed. Check valves and flap gates should be maintained so that they will operate properly and close securely. Sand or other material that may have accumulated in or around flap gates so as to obstruct the flow or prevent functioning of the gates should be removed. Outfall ditches, bank slopes, or berms should be properly maintained in the vicinity of horizontal outfall pipes.

During each inspection special attention should be given to check valves, gaskets, and standpipes, and each component should be maintained in the same working condition as when it was initially installed.

The area in the immediate vicinity of the wells should be kept free from weeds, trash, and debris. Mowing and weed spraying should be extended at least 5 ft beyond the well and the ground shaped and maintained for inspection and servicing of the wells.

Appendix A is a description of a field relief well installation made with the specific purpose of proving up the design principles for relief wells.

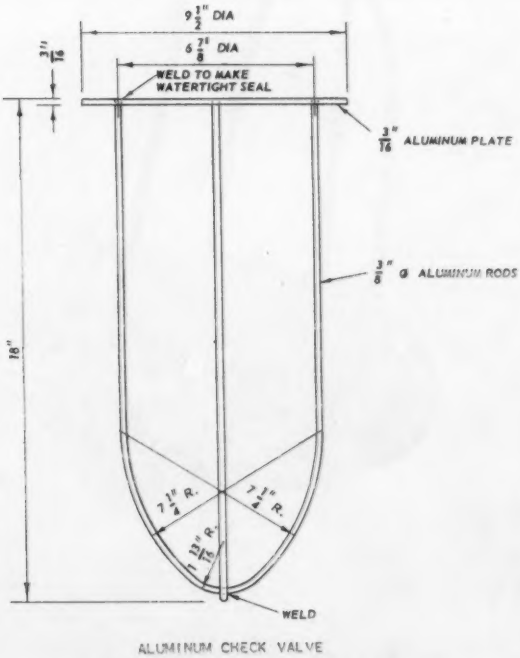


Fig. 5b. Details of check valve

Seepage Berms

Seepage berms can be constructed by hydraulic fill methods or by hauling. The method of construction and type of fill to be used will depend on whether the berm has been designed as an impervious, semipervious, pervious or sand, or a free-draining berm in accordance with the following tabulation.

Type of Berm	Type Soil	Berm Coefficient of Permeability (k_v)* x 10^{-4} cm/sec	Method of Placement	Lift Thickness in.
Impervious	Random	1	Hauling	12 to 18
Semipervious	Silty sand or sand	1 100	Hauling or dredging	--
Pervious	Sand	100	Hauling or dredging	--
Free-draining	Random and gravel filter blanket	**	Hauling	--

* Vertical permeability of a pervious stratum.

** Seepage-carrying capacity of gravel blanket equal to or more than 1 to 2 gpm per linear foot of berm with a head loss of less than 1 or 2 ft for flow through full width of berm.

Berms do not require any special compaction other than that resulting from placement and spreading operations. Special precautions must be taken in the construction of free-drainage berms to insure that the filter layers are properly constructed and the gravel filter has the required permeability.

Seepage berms require relatively little maintenance. They should be kept graded to drain, and any large gullies that develop should be filled. Objectionable growth of weeds or brush should be controlled by grazing or periodic mowing.

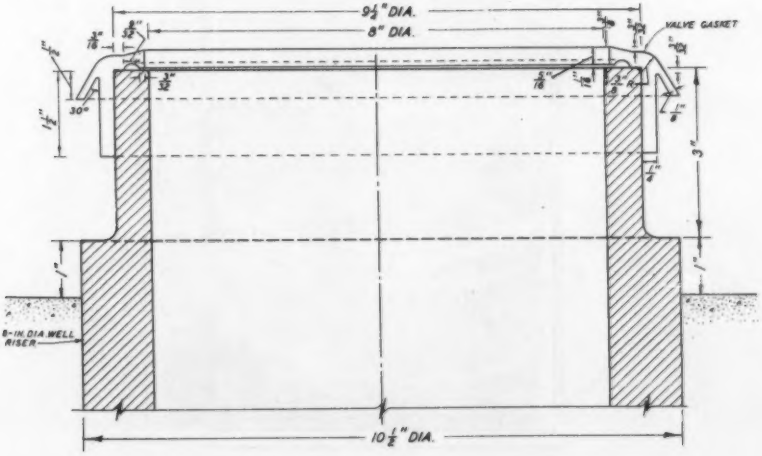


Fig. 5c. Details of rubber gasket

Riverside Blankets

Riverside impervious blankets should be constructed of the most impervious material readily available. It should be placed in 6- to 12-in. lifts and compacted with controlled movement of the hauling equipment or at least three coverages of a crawler-type tractor exerting a tread pressure of at least 6 psi. The thickness of the blanket will depend upon its permeability and other design considerations but should be a minimum of 3 to 5 ft.

Borrow for seepage berms, or for impervious riverside blankets or sublevees, should either be obtained at a distance of 1000 to 1500 ft or more from the riverside levee toe, or borrow operations should be controlled so as to leave a blanket of clay or silt at least 5 ft thick over the underlying pervious sands.

If the blanket will be subject to very severe scouring action, it should be protected by means of spur and/or abatis dikes strategically located. Borrow pits in which impervious blankets are placed should be drained to permit the growth of willows.

Borrow pits that have been excavated to sand for levee construction can be partially refilled with silt by construction of spur and abatis dikes at strategic locations along the levee. Although such a blanket of silt and sand will not be as impervious as a compacted clay blanket, it will materially reduce underseepage and hydrostatic pressures landward of a levee. (Construction of abatis dikes is subsequently discussed.) Collection of silt by means of abatis dikes requires upstream inlet ditches to bring water into the borrow pits at below bankfull stages and downstream outlet ditches to permit drainage of the

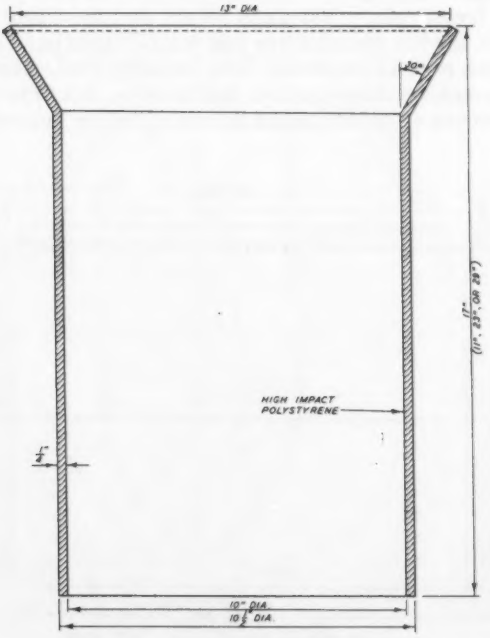


Fig. 6a. Details of standpipe

pits after the high water has receded. Drainage of the pits is required to permit the growth of willows which are necessary if the pits are to be filled by siltation. The growth of willows is necessary to replace the abatis dikes which have an expected lift of no more than 3 to 5 years.

Riverside blankets should be inspected after high water to check on scour. If the blanket has been damaged by scour, the scour should be repaired and the blanket protected by means of properly placed abatis dikes and by encouraging the growth of willows.

Sublevees

Sublevees constructed around critical seepage areas should have a minimum crown width of 5 ft and side slopes of 1 on 2-1/2 or 3. They should be built of relatively impervious soil compacted in 9-in. lifts with at least three coverages of a crawler-type tractor or the equivalent by controlled movement of hauling equipment. Borrow for sublevees should never be obtained between the levee and the sublevee. The material should be placed at approximately

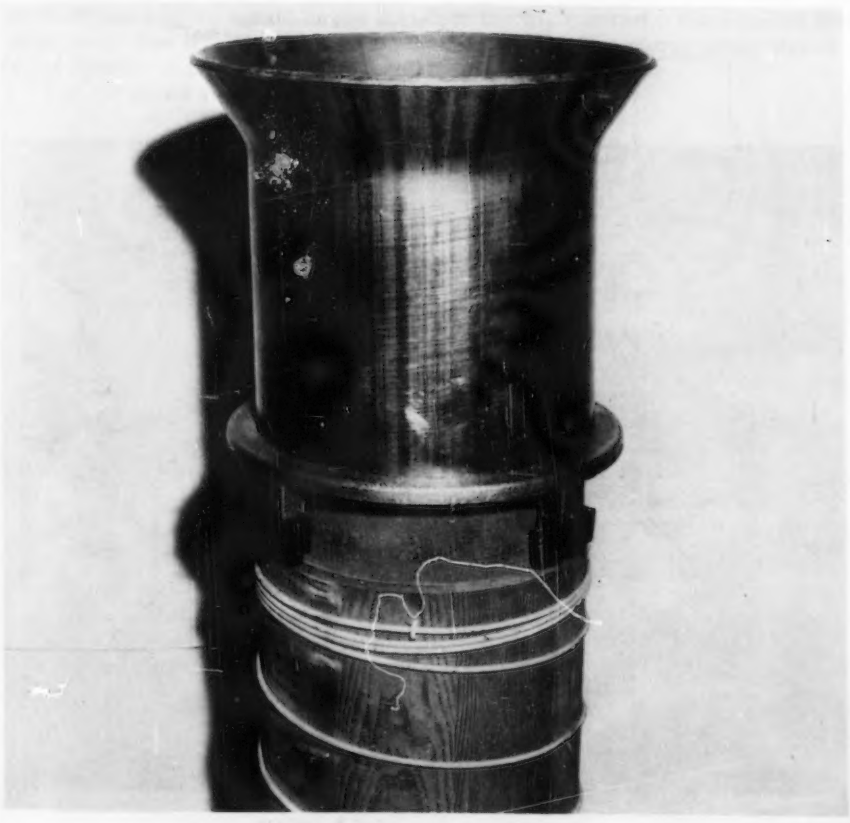


Fig. 6b. Standpipe in place on cast-iron tenon

optimum moisture to insure its stability when subjected to a head of water and to minimize the development of shrinkage cracks. The base for the sublevee should be properly prepared so as to prevent development of piping beneath the sublevee. Sublevee basins should be provided with paved overflow spillways to prevent crevassing in event of overflow as a result of seepage or surface runoff from the main levee. A gated outlet should also be provided to drain the basin after a high-water period. The sublevee basin should have at least 1 ft of freeboard above the maximum anticipated pool elevation with the spillway in operation. Sublevee basins longer than about 300 to 500 ft should be provided with cross dikes and each basin thus formed should be provided with an overflow spillway. The sublevees and dikes should be sodded and maintained.

Sublevees should be inspected periodically to check their grade and section, and the condition of the control gates, spillways, and drains. Sublevees and the area within them should be maintained free of objectionable weeds and brush, any erosion of the sublevees should be repaired, and control gates, spillways, and drains should be kept in operating condition at all times.

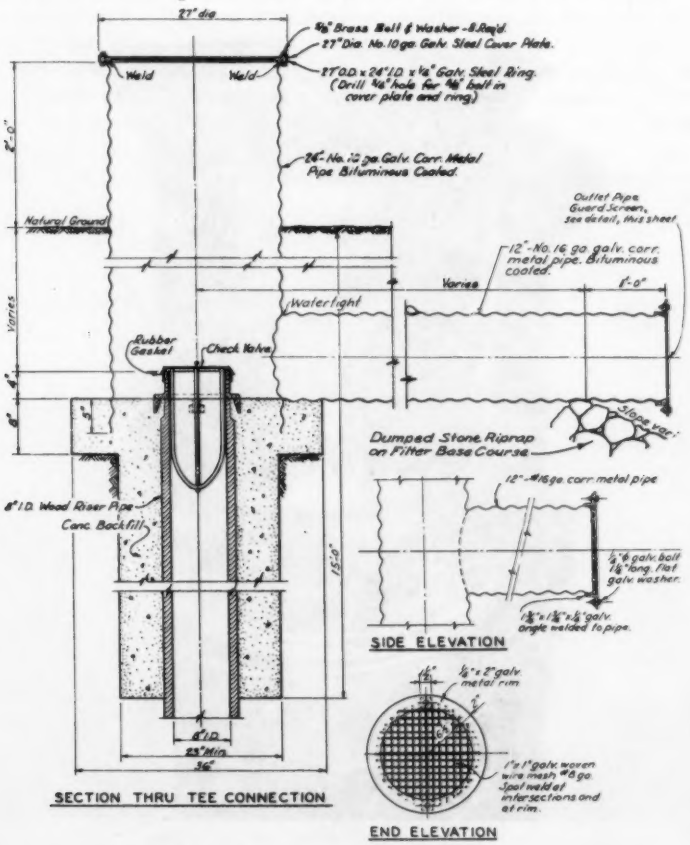

27" dia.
⅝" Brass Bolt & Washer - 8 Req'd.
27" Dia. No. 10 ga. Galv. Steel Cover Plate.
Weld
Weld
27" O.D. x 24" I.D. x ¼" Galv. Steel Ring. (Drill ⅝" hole for ⅝" bolt in cover plate and ring.)
2'-0"
24"-No. 12 ga. Galv. Corr. Metal Pipe Bituminous Coated.
Natural Ground
Outlet Pipe. Guard Screen, see detail, this sheet
12"-No. 16 ga. galv. corr. metal pipe. Bituminous coated.
Varies
1'-0"
Varies
Water tight
Rubber Gasket
Check Valve
Stone, var.
Dumped Stone Riprap on Filter Base Course
8" I.D. Wood Riser Pipe
Conc. Backfill
15'-0"
8" I.D.
23" Min.
36"
SECTION THRU TEE CONNECTION
12"-#16 ga. corr. metal pipe
½" ø galv. bolt 1½" long. flat galv. washer.
½" x 1½" x 1½" galv. angle welded to pipe.
SIDE ELEVATION
½" x 2" galv. metal rim
1" x 1" galv. woven wire mesh #8 ga. Spot weld at intersections and at rim.
END ELEVATION

Fig. 7. Tee-type well outlet

Abatis Dikes

Abatis dikes usually consist of a row of posts across the borrow pit that support 2- by 4-in. stringers. To these stringers are stapled field fencing and snow fence as shown in Fig. 9. The dike is braced with 2- by 4-in. bracing anchored to stakes driven into the ground on 18-ft centers. In order to prevent undercutting of the dike as result of scour, it should be protected both upstream and downstream by means of a riprap mattress.

Generally, abatis dikes vary in height from 4 to 8 ft. The length and depth of supporting posts are usually varied in proportion to the height of the dike as shown in Fig. 9. Prior to construction of a dike, minor grading and leveling is usually required in the bottom of the borrow pit along the dike line. Posts should be 6 in., either round or square, and should be sound and free of defects that would impair their effectiveness for the purpose intended. The snow fence usually consists of 1-1/2- by 12-in. slats, with a spacing of about 2 in. between laths. The field fencing should have top and bottom wires of not less than No. 9 gage wire, and bars and staves should be not less than No. 11 gage wire. The staves and bars should be spaced on about 6-in. centers. The width of the fencing should be not less than the width between the adjacent dike stringers. The fencing should also be zinc-coated. The riprap stone should be of durable quality and should grade from about 3 to 150 lb.

Fig. 10 shows a completed abatis dike.

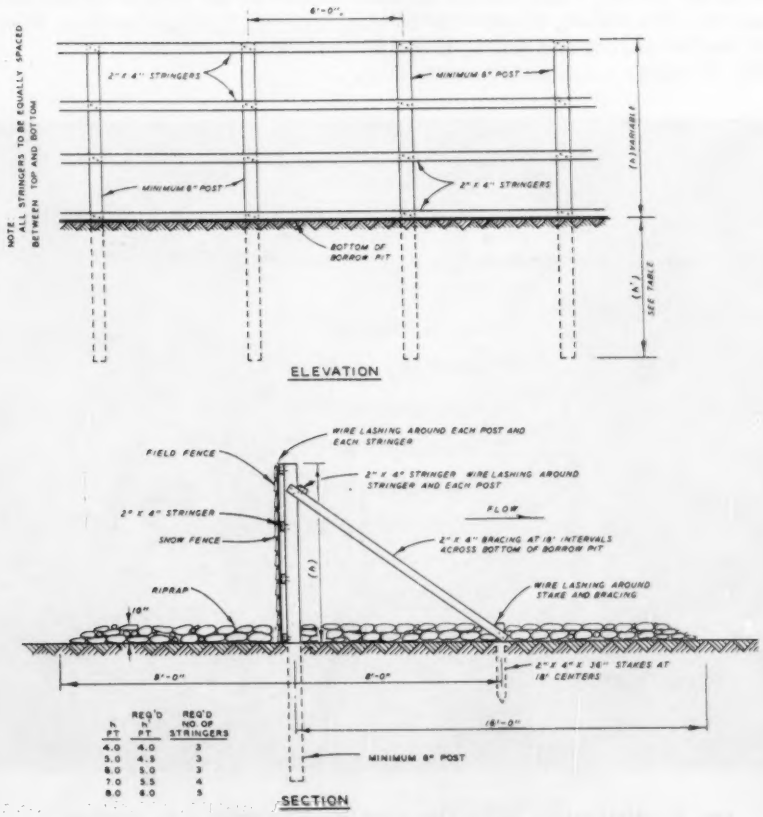
Fig. 8. Well guard and outlet used in areas subject to flooding by sanitary or industrial sewage

Drainage Trenches

Drainage trenches are excavated along the landside toe of a levee for the purpose of tapping the underlying pervious substratum. In order to be effective, the trench must penetrate any upper strata of clay, silt, or very fine sand. Although the depth of the trench will usually be set on the basis of previous borings, soil strata frequently vary in relatively short distances and therefore it may be necessary to adjust the depth of the trench during construction in order to insure that it taps the main underlying aquifer.

Drainage trenches are usually excavated immediately landward of the levee toe. Side slopes of such trenches should be as steep as possible (usually 1 on 1 or 1 on 1.5) but must be stable so as not to endanger the levee slope. The trenches should be constructed in the summer or fall when the water table is lowest; even then it may be necessary to use a dewatering system to achieve the required depth for the trench.

For a drainage trench to function properly, it is imperative that the filter layers be properly graded and be carefully placed. Likewise, the collector pipe should be properly installed. There should be no joints with openings



h FT	REQ'D h' FT	REQ'D NO OF STRINGERS
4.0	4.0	3
5.0	4.5	3
6.0	5.0	3
7.0	5.5	4
8.0	6.0	5

Fig. 9. Details of abatis dike

larger than the perforations in the pipe. Precautions should be taken to prevent the filter layers or collector pipe from becoming flooded with muddy surface water during construction.

The excavation for the trench may be refilled with material taken from the trench and compacted with either a crawler-type tractor or by hauling equipment.

Drainage trenches of the design previously discussed should be given the applicable inspection and maintenance described for relief wells.

Cutoffs

Where the depth of the pervious substratum is less than about 40 ft, an impervious cutoff can be constructed at the riverside toe of the levee by means of either open excavation or a trenching machine and backfilling with impervious soil. The cutoff should be placed either beneath the levee (in the case of new levees) or at the riverside toe. If placed at the riverside toe it should be tied into the levee by means of an impervious blanket.

Cutoffs may be constructed of earth, steel sheet piling, or possibly by grouting through grout holes on close centers; however, earth construction is considered the most practical method. Although steel sheet piling is commonly used for cutoffs, experience has indicated that it is not entirely watertight as leakage occurs through interlocks, at splices, and through torn interlocks. Unless the sheet pile cutoff is tight and continuous, little reduction of pressure landward of levees can be expected. However, a sheet pile wall properly driven will significantly reduce the danger of piping beneath the levee. The high cost of steel sheet piling generally precludes its use for deep cutoffs. Grouting of alluvial sands has been attempted but is quite expensive and at its

Fig. 10. Abatis dike across borrow pit, Columbia Levee District. St. Louis District, CE

present stage of development cannot be considered as a practical means of cutting off a deep pervious aquifer.

An open excavation for a cutoff should be cut on slopes which will insure stability of the slopes and the levee. A wellpoint system will usually be required where the excavation extends below the water table. The excavation should be backfilled with impervious soil (clay or silt) compacted in layers by means of a crawler-type tractor or controlled movement of hauling equipment.

A cutoff can also be constructed by dragline or specially built trenching machine capable of digging trenches 40 to 50 ft deep. In this method of construction the trench is held open by keeping it filled with a clay slurry. It is essential that the clay content of the slurry be sufficient to coat and seal the walls of the trench in order that the hydrostatic pressure can be effective in holding the trench open. The clay slurry can be prepared by chopping the clay with a disk harrow and mixing it with water by means of pumps and/or paddles. The trench is then filled with the slurry. In order that the procedure be successful, the operations of digging, keeping the trench filled with slurry, and backfilling must all be carried out simultaneously.

Where a levee is underlain by a relatively thin deposit of natural levee or crevasse sand, in turn underlain by impervious strata, the sand stratum can be cut off by either of the above procedures.

Cutoffs require no inspection or maintenance.

Piezometers

Probably the most positive way to determine the effectiveness of any seepage control measure is the installation of an adequate system of piezometers.

Piezometers generally consist of all-brass or bronze screens set in filter sand, a riser pipe generally extending 3 ft above ground surface, and a cylinder of concrete around the riser pipe extending from just below the piezometer cap to 30 in. below ground surface. The piezometer screen normally is 24 in. long with a 40-mesh screen or No. 18 or 25 slot size. The riser is generally commercial grade, standard weight, 1-1/4-in.-ID galvanized steel pipe with couplings of galvanized steel, except that a plastic coupling should be used between the brass screen and riser pipe. The riser pipe should be fitted with a standard-weight galvanized steel cap, with a 1/8- or 1/4-in. hole in its top, attached to the concrete cylinder around the riser pipe. All joints should be treated with a suitable joint compound which will seal the joint and completely cover and protect that part of the riser pipe from which the galvanizing has been removed by threading. All joints should be made tight but care should be taken not to damage the plastic coupling while tightening the assembly.

Piezometers should be installed in 6-in. minimum diameter holes drilled to the required depth. The holes must remain open, or be held open with casing if necessary, to allow placement of the sand filter around the piezometer screen and to allow proper backfilling around the riser pipe.

Samples should be taken at every change in type of soil and at intervals not to exceed 3 ft, in order to obtain representative samples of the various strata encountered.

The filter material should consist of clean, well-graded medium to fine sand which should extend 6 in. below the bottom of the screen, completely surround it, and extend at least 6 in. above the top of the screen.

Space around the riser pipe should be backfilled with impervious soil or concrete from the top of the filter material, or above where sand caves in when casing (if used) is withdrawn, to within 30 in. of the ground surface. Soil backfill should be thoroughly compacted. If the space around the riser pipe contains water, it should be filled with concrete placed by the tremie method.

After installation is completed, each piezometer should be pumped until a clear stream of water is obtained. If the piezometer screen is above the water table or the water table is too deep to permit pumping, the piezometer should be filled to the top with clear water and the rate at which the water falls in the riser pipe recorded. Depth to water should be recorded every minute for the first 5 min and at 5-min intervals thereafter for the following periods, depending on the soil in which the screen is set.

Piezometer Screen Set In	Period of Observation (min)	Approximate Time for 50% Fall (min)
Sandy silt	30	30
Silty sand	10	5
Fine sand	5	1

If the piezometer has been installed properly, the time required for the water in the riser pipe to fall 50% of the total fall to the groundwater table should not be appreciably, if any, greater than that listed above.

All piezometers should be inspected annually for damage or any unusual condition that might affect their performance. The site of piezometers should be kept clear of weeds and brush and cared for in the same manner as described for relief wells. Any damage to or maintenance performed on piezometers should be reported, and the piezometers repaired.

SUMMARY

It is obvious that the selection of the proper method for the relief of underseepage and attendant excess pressures will depend entirely upon the existing foundation conditions with due cognizance being taken of economy and safety.

ACKNOWLEDGMENT

The seepage investigations described in this paper were conducted by the Waterways Experiment Station with the assistance of the Memphis, Vicksburg, and New Orleans Districts, CE, under the general direction of the Mississippi River Commission. The authors actively participated in the study from its inception. Mr. R. I. Kaufman, Jr. Member ASCE, assisted with analysis of the data.

APPENDIX A

Field Study of Control of Underseepage by Relief Wells

A part of the general study of methods for controlling underseepage along levees in the Lower Mississippi River Valley included the installation and testing of a full-scale relief well system at Trotters 54.(5)

The primary purposes of installing the relief well system at Trotters were to make a full-scale field test of the efficacy of a relief well system and to obtain more knowledge concerning the action of relief well systems in general. Other purposes of the study were:

a. To determine the distribution and amount of hydrostatic pressure in the pervious substratum with and without relief wells in operation, including the head between the wells and landward of the well system.
b. To estimate the "effective" source of underseepage and the amount of underseepage with and without relief wells in operation.
c. To determine the effect of different well spacings on the reduction of substratum pressure and well flow.

Observed Hydrostatic Pressure

During the 1950 high water, the river stage reached an elevation of about 13.8 ft above the average ground surface landward of the levee and 15.7 ft above the water elevation in a drainage ditch paralleling the toe of the seepage berm. The maximum hydrostatic pressure observed at the toe of the seepage berm was 3.0 ft above the average ground surface or 4.9 ft above the tailwater in the drainage ditch, or about 22% and 31% of net head (H), respectively.

At the 1950 crest, excess pressures above the ground surface were observed as far as 3500 ft landward of the toe of the seepage berm. Data obtained from a line of piezometers perpendicular to the levee indicated that most of the seepage passing beneath the levee was rising to the surface in a strip about 300 ft wide along the toe of the seepage berm.

Design of Well System and Appurtenances

Well System

Criteria of Design.—The experimental well system at Trotters was designed for two different cases. Case A was based on the assumption that either all flow from the well system would be pumped over the levee or natural flow conditions would be such that the water in the collector ditch would not rise about el 178 and that the artesian head landward of the levee would not rise above the ground surface (el 180). Thus, little natural seepage would develop landward of the wells and it could be assumed that the landward top stratum would be analogous to an impervious top stratum case. For this case the maximum allowable head between wells P would be 8.4% H.

Case B was based on the assumption that the flow from the wells would not be pumped and that the tailwater over the wells would be at el 180 and that the net head between wells would not be more than 2 ft above the natural ground surface, or 7.2% H. In this case the top stratum landward of the wells was assumed to be only relatively impervious $k_b = 2 \times 10^{-4}$ cm/sec, where k_b equals vertical permeability of top stratum.

Most formulas and model test data available for the design of relief well systems are based on an infinite line of wells. In the design of the Trotters relief well system it was decided to make the system only long enough (1450 ft) to provide a central section of about 1000 ft which would be affected relatively little by end effects. The well system as finally designed consisted of 30 wells spaced on 50-ft centers.

Basis of Design.—The following assumptions and values were used in designing the Trotters relief well system:

a. Effective well radius r_w = 10 in. = 0.833 ft.
b. Infinite line source of seepage, parallel to an infinite line of relief wells, assumed in computing well flow and head between wells. End effects were considered in computing total flow from system.
c. Distance from source of seepage to line of wells s = 900 ft.
d. Thickness of top stratum z_b = 10 ft.
e. Thickness of pervious stratum d = 90 ft.
f. Permeability of pervious stratum $k_f = 1250 \times 10^{-4}$ cm/sec.
g. Penetration of well screen based on 50% penetration of principal pervious aquifer.

The formulas and model data used in computing the performance of the Trotters well system for case A were Jervis' design curves[2] and WES model A-a-1 (see reference (3), particularly Figs. 11 and 12, and reference (4)). Computation of the performance of the system for case B was based on WES model A-a-2.[3,4] All model data were adjusted to the foundation conditions existing at Trotters and the wells used at Trotters.

Design Computations.—All values given in the following paragraphs pertaining to computation of well flow, landward pressure, and seepage are based

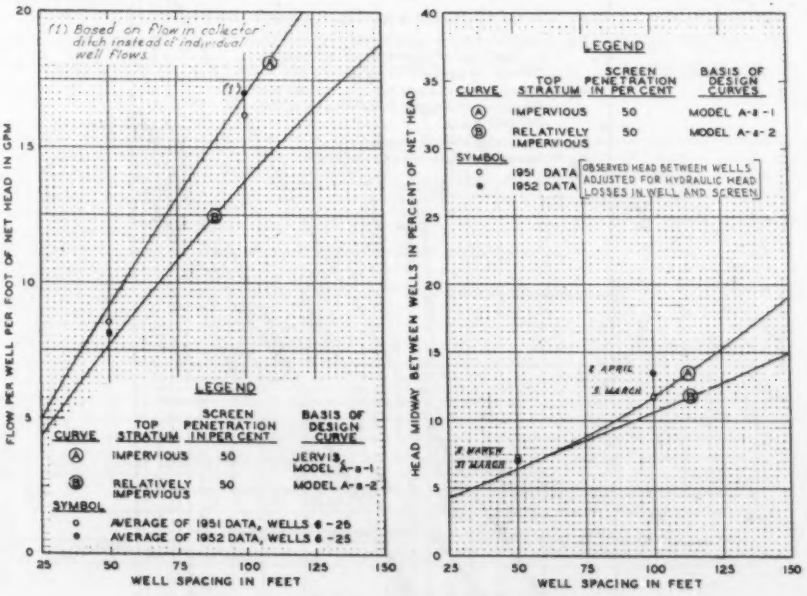


Fig. A1. Well flow and head between wells

on an infinite line of wells unless otherwise stated. The performance of the Trotters well system was computed for well spacings of 50 and 100 ft and a screen penetration of 50% as described below.

Case A.—For W = 0.50 and a = 50 ft, Q_W was computed from Jervis' curves to be 9.0 gpm per ft H and 9.0 gpm from WES model A-a-1 adjusted to the Trotters site (where W is the penetration of the well screen into the pervious aquifer expressed as a decimal, a represents well spacing in feet, and Q_W represents flow from a single relief well per unit of time). P was computed from Jervis' curves to be 4.1% H and 6.2% H from WES model A-a-1.

Case B.—Well flow, natural seepage (no wells), natural seepage (with wells), and head between wells all as computed from WES model A-a-2 are as follows:

Q_W = 7.6 gpm per ft H. P = 6.2% H

Natural seepage (no wells) $\frac{Q_S}{H}$ = 6.6 gpm per 50 ft of levee

Natural seepage (with wells) $\frac{Q_{S-W}}{H}$ = 1.0 gpm per 50 ft of levee

Total flow (well flow plus seepage), Q_{S+W} = 8.6 gpm per ft H per 50 ft of levee

Increase in total flow due to wells = 29%

The design curves for head between wells and well flow are shown for cases A and B on Fig. A1. The curves for P/H do not include any screen or

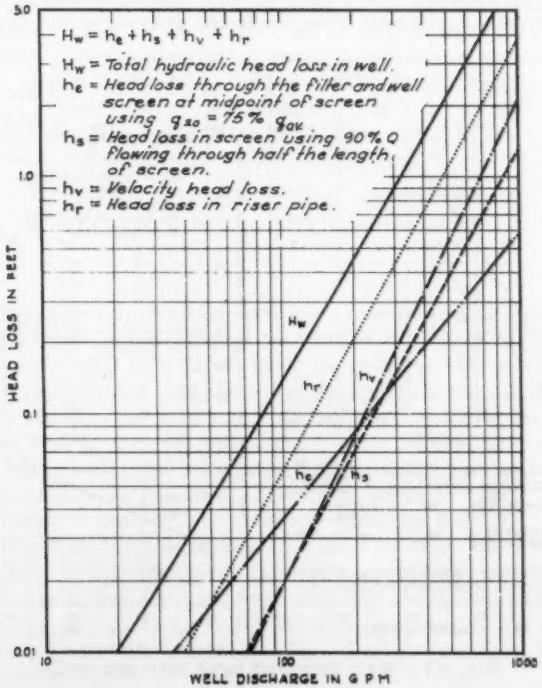

$H_w = h_e + h_s + h_v + h_r$
H_w = Total hydraulic head loss in well.
h_e = Head loss through the filter and well screen at midpoint of screen using q_{zo} = 75% q_{av}.
h_s = Head loss in screen using 90% Q flowing through half the length of screen.
h_v = Velocity head loss.
h_r = Head loss in riser pipe.
HEAD LOSS IN FEET
WELL DISCHARGE IN G P M
10
100
1000
1.0
0.1
0.01
H_w
h_r
h_v
h_e
h_s

Fig. A2. Computed head losses in wells for natural flow condition

hydraulic head losses in the well. The performance of the wells near the ends of the system was expected to be influenced by end effects but the computed Q_w and P/H were considered applicable to the central portion of the system.

Screen entrance and hydraulic head losses in the well were computed from Fig. A2 and added to the computed head between the wells as follows:

	Case A	Case B
Head on well system for project flood	29.7 ft	27.7 ft
Well flow, Q_w	300 gpm	240 gpm
Computed P	2.1 ft	2.0 ft
Entrance and well loss, h_w	0.9 ft	0.5 ft
Total head between wells (project flood)	3.0 ft	2.5 ft
Head between wells	7 to 10% H	7 to 9% H

Well Screen, Gravel Filter, and Riser Pipe.—The well screens consist of 6-in.-ID redwood pipe slotted with 3/16- to 1/4- by 2-in. slots. The slots have an open area of 18 sq in. The screen portion of each well is 40 ft long. It is surrounded with a 6-in. layer of filter gravel (see Fig. A3). The design of the filter was based on the filter criteria previously given. The filter has an estimated permeability of about 5000 x 10^{-4} cm/sec. Details of the well are shown in Fig. A4.

The tops of the riser pipes are capped with brass flap valves which may be used to close the wells or throttle the flow if desired.

Appurtenances

Collector Ditch.—The collector ditch for the wells is about 3.5 ft deep. It is paved with porous concrete underlain with a 2-in. blanket of well-graded coarse sand (see Fig. A4). The paving has a thickness of 6 in. in the bottom of the ditch and 4 in. on the side slopes. The ditch has a capacity of about 35 cfs.

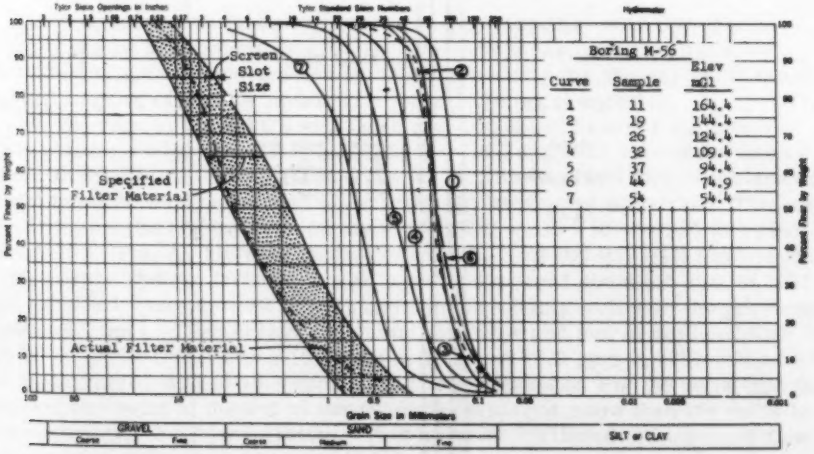


Fig. A3. Foundation sands and filter gravel

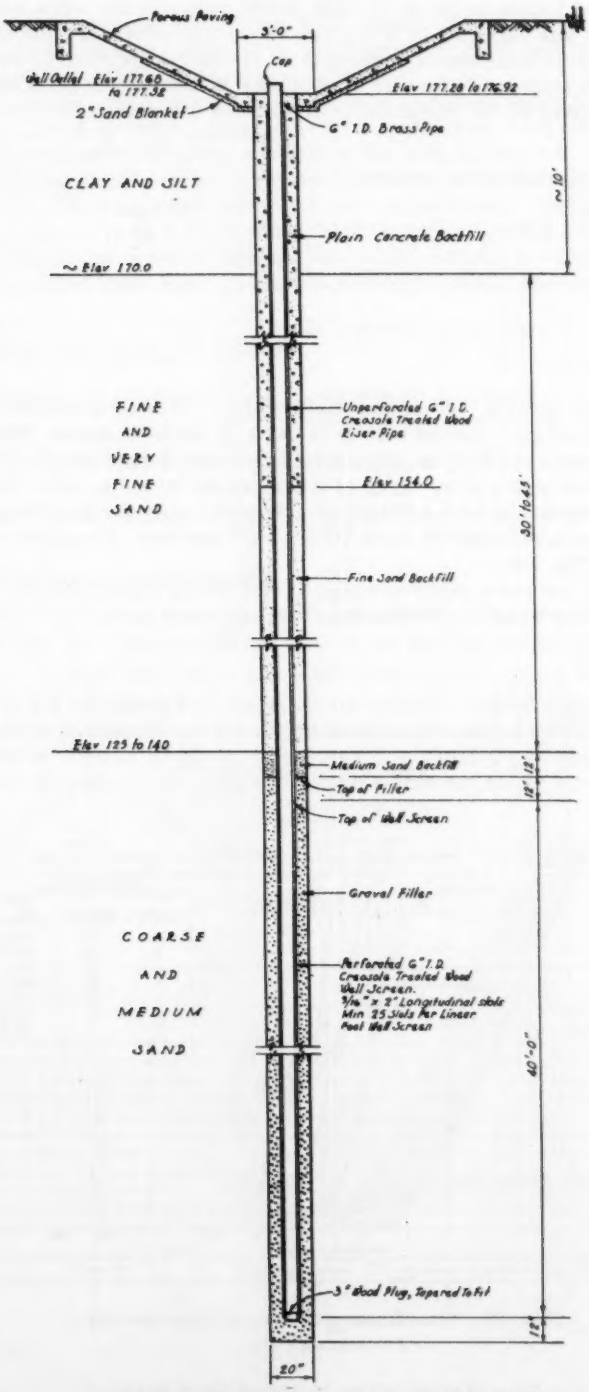


Fig. A4. Relief well and collector ditch

Control Culvert.—A control culvert through the sublevee at the downstream end of the well system consists of a 30-in. corrugated pipe and a 101 Calco gate.

Installation of Wells and Pumping Tests

The holes for the wells at Trotters were made by the reverse-rotary method.

After development of the well by surging, a pumping test was run to determine the flow for various drawdowns in the well. The wells had specific yields ranging from about 90 to 110 gpm. No sand was observed in the effluent from any of the wells during the pumping tests.

Analysis of Well Flow and Piezometer Data

Operation of the relief well system was observed during high-water periods in 1951 and 1952. The maximum head on the well system was about 7.5 ft in 1951 and 10.2 ft in 1952. During both high waters the well system was operated on both 50- and 100-ft centers; also, during each high-water period the wells were completely closed and substratum pressures and seepage flows were measured. Piezometer readings and well flows were observed at frequent intervals for different well operating conditions.

The general operating procedure was to allow the system to operate with the wells on 50-ft centers up to about crest stage, at which time every other well was closed so as to create a system with wells on 100-ft centers; after obtaining well flow measurements and piezometer readings for both of these conditions, the well system was closed and piezometer readings and seepage measurements were made. After obtaining these measurements, all wells were then reopened.

Well Flow

The individual well flows were measured by a special well flow meter which could be lowered into the wells. The flow in the collector ditch was measured at the downstream end by cross sectioning the stream and measuring the velocity of flow by means of a midget Gurley flow meter.

The average of individual well flows per foot of net head for all data obtained during rising river stages during the 1951 and 1952 high-water periods is shown by Fig. A5. (Well flows for a = 100 ft during 1952 high water are not shown because well meter was apparently not functioning properly.) The average flows per foot of net head on the system for wells 6 through 25 are also tabulated in Fig. A5 for well spacings of both 50 and 100 ft. Well flows in the center of the system for a 50-ft well spacing averaged about 8.5 gpm in 1951 and about 8.1 gpm per well in 1952 and compare fairly well with the predicted flow of 9.1 gpm. The average individual well flow per foot of net head with the wells on 100-ft centers was 16.2 gpm in 1951 and 16.9 gpm in 1952 compared to the predicted flow of 16.8 gpm. The bar diagrams on Fig. A5 show that in general the wells at the end of the system discharged more than the wells in the central portion of the system. This trend is attributed to increased flow caused by end effects.

The relationships between well flow and net head on the well system are shown in Fig. A6 for the 1951 and 1952 high-water periods. It can be seen that the 1951 and 1952 well flows are similar up to a net head of about 5 ft, and that the well flow varies linearly with net head up to about 5 ft on the well system. For net heads greater than 5 ft, the 1951 well flows increased at a rate greater than the rate of net head increase; and after the crest of the flood had occurred, greater well flows were obtained at a given head than the flows occurring under the same head before the crest had occurred. The 1952 data indicate a nearly linear relationship between flow and net head for the heads experienced during rising river stages. As in 1951, after the crest of the flood had occurred, greater flows resulted under the same head. Inasmuch as the flow from the relief well system should vary linearly with the net head on the system, provided the distance to the effective source of seepage remains constant, the occurrence of a greater flow under a given net head after the crest of the flood has passed is attributed to a decrease in the distance to the effective source of seepage.

Substratum Pressure

Piezometer gradients along line M for the 1952 high-water period with and without the relief well system in operation are shown in Fig. A7. The piezometric data plotted on this figure show that very little substratum pressure developed along or landward of the line of wells when all wells were open. Only in a few isolated areas landward of the well systems was there any excess head above the ground surface. A prime reason for there being no head above the ground surface was that the wells discharged at an elevation below the natural ground surface, and no ponding occurred in the outfall ditch which conducts the well flow away from the levee. When the wells were closed, the substratum pressures rose rapidly, and the net hydrostatic pressure in the substratum was between 40 and 50% H.

River stages, the hydrostatic pressure along the line of wells when closed, the average ground surface and the piezometric surface between wells for spacings of 50 and 100 ft, and the elevation of the water surface in the collector ditch all are shown for certain selected days during the crest of the 1952

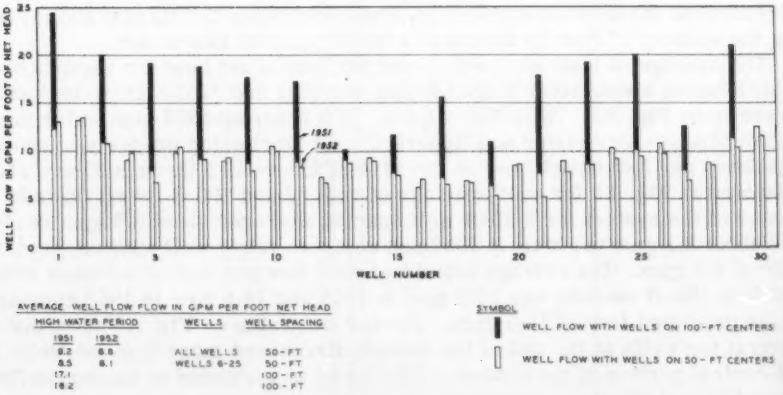


Fig. A5. Individual well flow per foot of net head (1951 and 1952)

high water in Fig. A8. The head midway between wells along the system as shown in Fig. A8 was plotted in Fig. A9 for well spacings of 50 and 100 ft. The observed data shown in this figure were corrected for well losses.

Seepage Control

The efficacy of the relief well system in controlling seepage was demonstrated by the absence of sand boils and/or excessive seepage landward of the well installation during the 1951 and 1952 high-water periods. Again it should be noted that the efficiency of the system is increased greatly by virtue of the landside drainage conditions and because the wells discharge at an elevation somewhat lower than the natural ground surface. A comparison of seepage conditions both upstream and downstream from the well system further indicates the ability of the system to control seepage. The following summary of seepage conditions was abstracted from field notes taken during operation of the well system in the two high-water periods.

While the well system was in operation with wells on either 50- or 100-ft centers, no sand boils were observed in the drainage ditch landward of the levee toe adjacent to the well system; however, inspection of this ditch upstream and downstream from the well system disclosed the presence of several pin boils in the bottom of the ditch for distances of about 1000 ft upstream and downstream from the ends of the well installation. No seep water was observed landward of the well system when it was in operation, but seep water was observed standing in a field 600 to 900 ft landward of the levee toe at a point about 500 ft downstream from the relief well system. These conditions were observed for river stages from about 5 to 11 ft above the average ground surface.

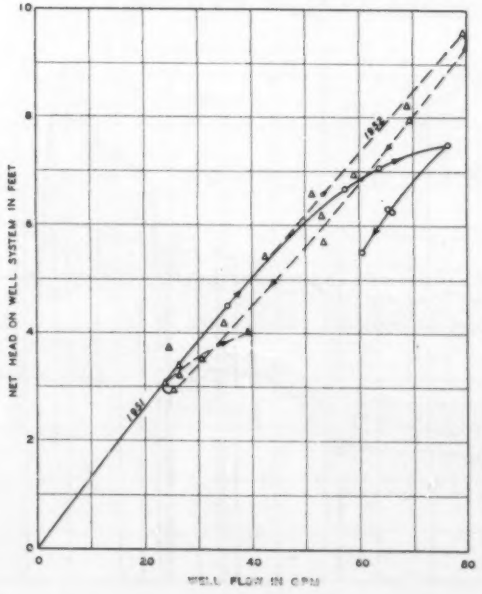


Fig. A6. Average individual well flow, a = 50 ft (1951 and 1952)

After the river crested at a stage of approximately 10.5 ft above the elevation of the water surface in the landside ditch in 1952, all relief wells were closed. Numerous sand boils became active in the outfall ditch for a distance of 1100 ft landward of the installation and in the drainage ditch paralleling and landward of the levee. Seepage into the ditch was 18.2 gpm per 100 ft of levee. Upon reopening all the relief wells (with H = about 9.3 ft) sand boils previously noted in the outfall and landside drainage ditches became inactive and no seepage was noted along the sublevee or flowing into the drainage ditch landside of the levee.

It is apparent from these seepage observations that the well system is effective in controlling sand boils and seepage landward of the well system for the stages experienced. The piezometer readings together with the observations made during the 1952 high water indicate that the well system not only reduced the substratum pressures but also intercepted a large portion of the natural seepage which otherwise would have risen to the surface landward of the levee.

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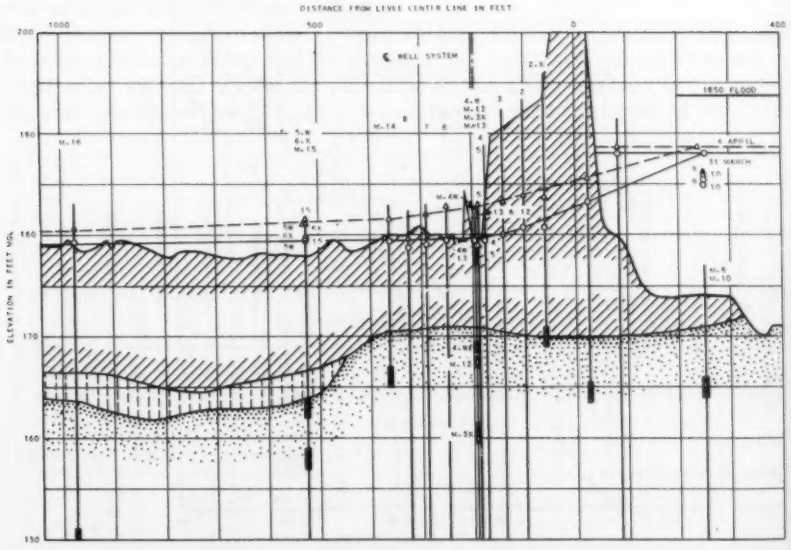

Fig. A7. Piezometer gradients, line M, 1952

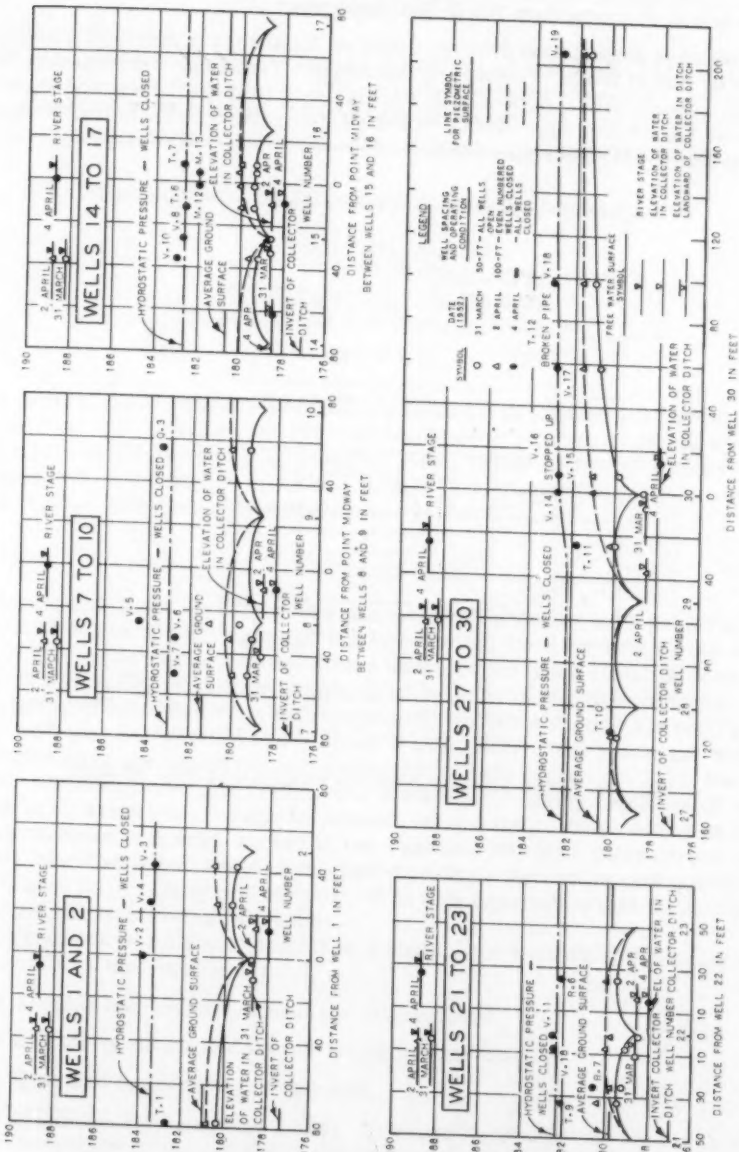
WELLS 1 AND 2
WELLS 7 TO 10
WELLS 14 TO 17
WELLS 21 TO 23
WELLS 27 TO 30
HYDROSTATIC PRESSURE — WELLS CLOSED
AVERAGE GROUND SURFACE
ELEVATION OF WATER IN COLLECTOR DITCH
INVERT OF COLLECTOR DITCH
RIVER STAGE
2 APRIL
31 MARCH
4 APRIL
WELL NUMBER
DISTANCE FROM WELL 1 IN FEET
DISTANCE FROM POINT MIDWAY BETWEEN WELLS 8 AND 9 IN FEET
DISTANCE FROM WELL 30 IN FEET
ELEVATION OF WATER SURFACE IN FEET MSL

Fig. A8. Hydrostatic head between wells (1952)

4. Waterways Experiment Station, Corps of Engineers, Relief Well Systems for Dams and Levees on Pervious Foundations, Model Investigation. Technical Memorandum No. 3-304, November 1949.

5. Waterways Experiment Station, Corps of Engineers, Control of Underseepage by Relief Wells, Trotters, Mississippi. Technical Memorandum No. 3-341, April 1952. Appendix B, "Analysis of 1952 Well Flow and Piezometer Data," and Appendix C, "1953 Pumping Tests," Technical Memorandum No. 3-341, February 1954.

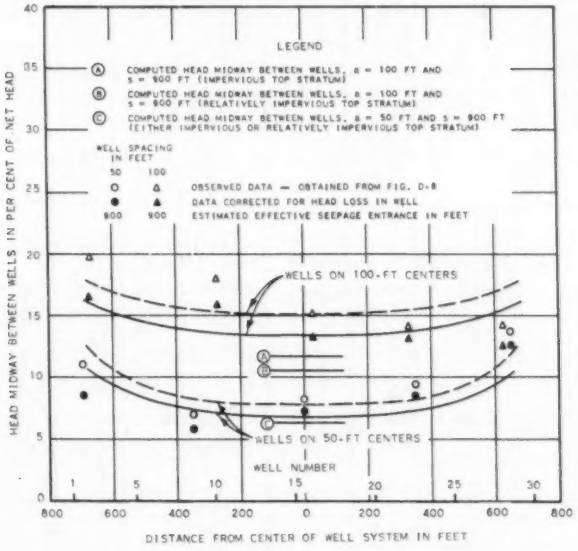
LEGEND
COMPUTED HEAD MIDWAY BETWEEN WELLS, a = 100 FT AND s = 900 FT (IMPERVIOUS TOP STRATUM)
COMPUTED HEAD MIDWAY BETWEEN WELLS, a = 100 FT AND s = 900 FT (RELATIVELY IMPERVIOUS TOP STRATUM)
COMPUTED HEAD MIDWAY BETWEEN WELLS, a = 50 FT AND s = 900 FT (EITHER IMPERVIOUS OR RELATIVELY IMPERVIOUS TOP STRATUM)
WELL SPACING IN FEET
50 100
OBSERVED DATA — OBTAINED FROM FIG. D-8
DATA CORRECTED FOR HEAD LOSS IN WELL
900 900 ESTIMATED EFFECTIVE SEEPAGE ENTRANCE IN FEET
WELLS ON 100-FT CENTERS
WELLS ON 50-FT CENTERS
WELL NUMBER
HEAD MIDWAY BETWEEN WELLS IN PER CENT OF NET HEAD
DISTANCE FROM CENTER OF WELL SYSTEM IN FEET

Fig. A9. Head midway between wells along well system

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PERMAFROST ASPECTS OF HUDSON BAY RAILROAD[a]

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ABSTRACT

A general description of the factors conducive to causing the condition of permafrost, the possible extent of this condition in Northern Canada and its effect on construction and maintenance. Also, recommended practices to contend with problems permafrost presents in connection with such projects, in particular, the Hudson Bay Railroad.

Recent increased activity in the north brought about by such projects as construction of the DEW Line and the Mid-Canada Line, as well as the relocation of the Arctic community of Aklavik has focused much attention upon the subject of permafrost and the problems of building in permafrost areas. It is interesting to note, however, that perpetually frozen soil, now termed "permafrost," was encountered and contended with nearly fifty years ago when construction of the railway was commenced to Hudson Bay. In 1939, the well-known Arctic explorer, Dr. Vilhjalmur Stefansson, making inquiries on behalf of the U. S. Army with respect to permafrost and its probable effect on railway operations in Alaska, turned to the Canadian National Railways for information based on actual experience gained during construction and operation of the Hudson Bay Railway to Churchill. It is this experience that forms the basis for the material in this paper.

Permafrost is not a material, it is a condition. Factors contributing to the occurrence of permafrost are:

1) Altitude and average annual air temperature.

2) Surface covering and its insulating properties.

3) Type and conductivity of sub-soil.

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4) Topography and natural drainage, both surface and sub-surface.

5) Location in relation to rays from the sun.

6) Precipitation.

With local vagaries excepted, permafrost may be expected in regions where the mean annual temperature is a few degrees below freezing - say 25° F. No direct correlation has been found between the presence of permafrost and any particular temperature isotherm, nor can the southern boundary of permafrost be clearly defined but as a rough guide, the southern limit of permafrost in Canada may be assumed to exist as shown in Fig. 1. It should be noted that nearly half of the total area of Canada, including the Arctic Islands, may be underlain by permafrost. The importance of studying this condition in connection with construction and maintenance of engineering facilities in the Canadian Northland cannot therefore be over-emphasized.

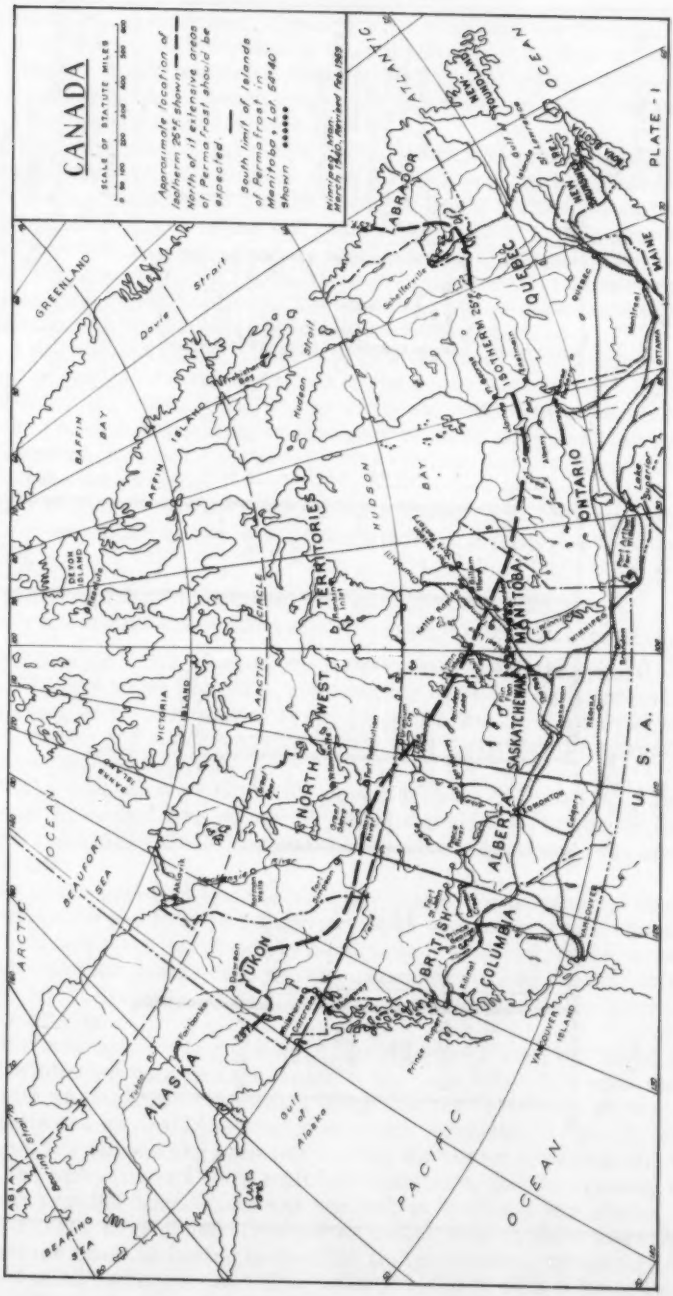
Construction of the Hudson Bay Railway was begun from The Pas, Manitoba, in 1910, to connect the Western Prairies with a seaport on Hudson Bay. The Pas is 340 miles north of the International Boundary between the United States and Canada. Railway mileages are from zero at The Pas, total distance to the port at Churchill is 510 miles.

Observations made during construction of the railway showed permafrost to occur in scattered patches or islands as far south as Wabowden, latitude 54°50'. Northward, these islands increase in extent until the Limestone River, latitude 56°30', is reached. From there to Churchill, latitude 58°47', the sub-surface is almost entirely permafrost.

This northerly section traverses the geological formation known as the Hudson Bay Lowlands, a vast area of muskeg, with very little timber, which the Indians call the "Land of Little Sticks"; the "Barren Lands" extend into this territory. Where the subformation is of organic materials covered with surface mosses and there is no accumulation of water, the summer thaw does not penetrate below a depth of 18 inches. On the few low ridges, where materials are fairly dry - clay, gravel or sand - and the covering of vegetation is light, the ground may thaw during the summer to a depth of 5 or 6 feet and under water to 10 feet or even deeper. Fig. 2 shows results of test borings at four principal stations on this Railway. At Churchill, under Lake Rosabelle, frost was not found until borings reached a depth of 42 feet, and in the harbour no frost was encountered within the limits of low-water mark.

The treeless tundra, immediately south of Churchill, is illustrative of the type of country most conducive to the occurrence of permafrost. The average yearly air temperature is 19°F. The area in general is very flat - an elevation of between 50 and 200 feet above sea level - there are numerous shallow lakes and ponds and between them are muskeg hummocks of 5 to 10 feet in height. The entire ground area is covered with a heavy vegetation of mosses, under them are organic materials to a depth of 4 to 18 feet, then glacial drift - grey clay, of fine texture and boulders. Lenses of clear ice, up to 6 inches thick, are found in the layers of both organic materials and drift. The higher ground (hummocks) is entirely frozen excepting for 12 to 18 inches below the surface; whereas the lower ground (hollows), where water accumulates in summer, thaws to a somewhat greater extent.

About one million years ago, the Keewatin ice cap covered this area and extended over the Prairies southerly into North Dakota, depressing the "Lowlands." During and after the last ice retreat, 7,000 to 15,000 years ago,


CANADA
SCALE OF STATUTE MILES
Approximate location of Isotherm 25°F shown — — North of it extensive areas of Permafrost should be expected.
South limit of Islands of Permafrost in Manitoba, Lat 54°40' shown ••••••
Principal Data Revised Feb 1959
PLATE - I
ATLANTIC OCEAN
LABRADOR
NEWFOUNDLAND
GREENLAND
Davis Strait
BAFFIN BAY
BAFFIN ISLAND
Hudson Strait
ISOTHERM 25°F
QUEBEC
MAINE
ONTARIO
HUDSON BAY
TERRITORIES
MANITOBA
NORTH WEST
SASKATCHEWAN
ALBERTA
U. S. A.
VICTORIA ISLAND
ARCTIC
BEAUFORT SEA
ARCTIC OCEAN
YUKON
BRITISH COLUMBIA
ALASKA
VANCOUVER ISLAND
Gulf of Alaska
PACIFIC OCEAN
BEARING SEA
ASIA

WABOWDEN
137
54°40'
28°F.
DEC. 26-1926
PEAT 4.0'
BLUE CLAY AND ICE 7.0'
ICE 0.1'
BLUE CLAY AND ICE 2.9'
BLUE CLAY
BOTTOM OF EXCAV.
FROST 1.8'
NO FROST 0.5'
FROST 11.5'
13.5'
BOTTOM OF FROST
ILFORD
286
56°10'
26°F.
NOV. 29-1926
PEAT 7.0'
BLUE CLAY AND ICE 9.5'
S. P. P. HARD PAN AND BOULDERS 0.45'
SOLID ROCK
FROST 2.5'
NO FROST 0.5'
FROST 14.5'
17.5'
BOTTOM OF FROST
KETTLE RAPIDS
331
56° 25'
21°F.
APRIL 8 - 1927
PEAT 5.5'
BLUE CLAY AND ICE 20.5'
CLAY AND BOULDERS
ENTIRELY FROZEN 26.0'
CHURCHILL
510
58° 47'
17°F.
AUGUST 1929
PERMANENT FROST 95.0'
BED ROCK
ICE 0.3'
ROCK
ICE 0.5'
ROCK
136'
146'
TEST BORING MADE BY DEPT. TRANSPORT S.E. OF LAKE ROSABELLE DEPTH 230 FEET DRILLER REPORTED CONTINUOUS TROUBLE FROM FREEZING
LOCATION
H.B.R. MILEAGE
LATITUDE
AVER. YEARLY TEMP.
DATE
CANADIAN NATIONAL RAILWAYS
HUDSON BAY RAILWAY
DIAGRAM SHOWING RESULTS OF TEST BORINGS
SCALE:- VERT. 1"=5'
WINNIPEG, MAN. 20 MARCH 1940
PLATE - 2

the elevation of this area raised. Observations show that the elevation above sea-level is continuing to increase. It is interesting to note that several well preserved Walrus skulls and tusks were found in gravel being excavated for ballast from a low esker or beach ridge, 215 feet above and 50 miles inland from the present shoreline of Hudson Bay.

As this tundra is yet in a primitive era in relation to evolution of formation, it is suggested that, the surface may, through time, with improved natural drainage, transform to a more advanced state, perhaps to soil which would produce timber such as the fair stands of spruce presently growing on islands and river banks where there is good drainage. Where ditches have been excavated adjacent to the Railway, black organic materials are exposed in the former lake beds; grasses and weeds will grow in these materials with entirely different characteristics from the mosses of the tundra.

Fig. 3, a sectional diagram, illustrates how observed annual thawing, at water level, of the many lakes and ponds in the region causes over-hang at the shores which periodically fall and increase the water surfaces; this process may result in the joining of a number of lakes and ponds and may lead to the erosion of more and deeper water courses and thus improved drainage for the entire region.

The effect of permafrost in connection with construction and maintenance of the Hudson Bay Railway can best be described by considering the route in two sections: -

A) From the south limit of "patches" or "islands" of permafrost, northerly for a distance of 220 miles to where it becomes general.

Over this section which takes in the Nelson River drainage system, elevation 750 to 450 feet above sea level, the topography varies from extensive flat areas of muskeg to rolling country of the Canadian Shield - rock, with heavy overburden of clay and sand, the latter fairly well timbered with spruce, jack-pine, tamarac, aspen and birch.

Construction was carried out, on this section, from 1912 to 1915. Patches of permafrost were encountered at many locations between Wabowden, Mile 137, and the crossing of the Limestone River, Mile 352, in both clay and organic materials.

Drainage of the muskegs was the first operation. No mechanical equipment was available and to work horses or mules was impractical. Hundreds of miles of ditches were excavated by hand, frozen materials were taken off in layers of about six inches deep as they thawed but in many locations it was essential to effect drainage immediately, then permafrost was actually chopped out with axes.

Where the roadbed was in cut and fill, embankments were built with materials from adjacent cuts, but the length of haul was limited by what was practical with wheelbarrows and small dump cars equipped with double flanged wheels hauled over pole tracks manually, excepting in special locations where a few horses had been left in after the winter freight haul. The balance of the embankments were built with materials, mostly organic, excavated from parallel borrow pits and wheeled, in barrows, into place.

Timber bridges were constructed during winter season, when materials and pile drivers could be hauled in over the frozen terrain. Steam was employed to jet piles through frost. Five cedar piles up to 40 feet long, spaced 3 feet centre to centre in each bent, with distance between bents 14 feet.

'A'
RIVER
OCEAN
'A'
SHALLOW LAKE
POND
It would appear that areas of water are gradually increasing and perhaps, through time, result in improved natural drainage
PLAN
Elevation of area is generally highest at banks of main rivers
Permanently frozen from bottom of moss
"Hummocks", except surface moss completely frozen at all seasons
Overhang of banks cause "cave-ins" Annual thaw of water level appears to be increasing areas of lakes and ponds.
Top of permafrost
Organic materials
Lenses of clear ice
RIVER
Active layer is deeper under rivers and lakes
Clay and Boulders
SECTION 'A-A'
DIAGRAM TO ILLUSTRATE SUMMER CONDITIONS IN TUNDRA
SOUTH OF CHURCHILL, MAN.
NOT TO SCALE
WINNIPEG, MAN. 27 MARCH 1940
PLATE - 3

Track was laid each winter with a "Pioneer" tracklaying machine, when the unconsolidated roadbed was frozen, with temperatures down to 50 degrees below zero. The following summer, embankments were progressively improved with trainfill and the track was ballasted with pit run sand and gravel hauled considerable distances.

Trainfilling and ditching were continued each summer until 1917, when due to wartime conditions further work was suspended and trains were not operated north of Pikwitonei, Mile 214. Beyond there, without maintenance, embankments settled to such an extent that at some locations track was hanging in the air and sink-holes occurred where cuts had been excavated through knolls of frozen organic materials and ice. Removal of surface insulation and exposure to annual thawing caused water to accumulate and aggravate the situation.

In 1926, it was decided to complete the railway to the Bay and to construct a harbour with facilities for trans-shipment of goods, principally grain, between rail and ocean freighters. The initial requirement was to rehabilitate the 330 miles of track laid and facilities built prior to the suspension of construction. As it was customary to use untreated ties during construction, a large percentage had to be replaced and heavy programmes of ditching, trainfilling and rebuilding of timber bridges were necessary. Fortunately, draglines and other mechanical equipment had become available and they could be moved in by rail. Conditions were much easier than during original construction when all supplies had to be hauled in by teams during winter as there was no summer transport other than on foot.

Extensive surveys were carried out to plan more effective drainage. Draglines were moved out on pads as much as three miles to commence excavation of off-take ditches, up grade, to the track and then parallel side ditches. A large percentage of these excavations were of organic materials although some parts were through clay and occasionally solid rock. Much was permafrost, this was usually excavated in layers of from 6 to 12 inches deep as they thawed after exposure; however, in critical locations where the necessity for drainage was urgent, explosives were used, but efficiency factor of explosives in blasting frozen materials is comparatively low.

In connection with the ditching, hundreds of the original culverts were renewed and additional ones installed. Selection of culvert sites is of great importance; if it is at all possible they should not be placed on permafrost. Where this is not practicable, it is necessary to excavate well below grade and backfill immediately with dry material, preferably sand or gravel, before thawing begins. A mat of old bridge stringers, laid with a good camber, should then be placed as a foundation for the culvert.

For such locations, experience has shown the most servicable type of culvert to be a cedar box, with solid floor, constructed of 10" × 10" or 12" × 12" timbers. These structures are flexible and will function even if they become distorted. In northern latitudes, a life of thirty or more years may be expected for exposed untreated cedar; under water, it will last almost in perpetuity. Due to the possibility of settlement and other movement of a culvert placed on permafrost, use of treated timber would not appear to be economical. As an alternative, corrugated iron pipe is now available.

There are no unusual features in maintaining and operating this section of railway. It is important, however, that drainage ditches are re-excavated periodically, say every ten years, to prevent accumulation of water against the slopes of embankments. Cuts, particularly those excavated through

permafrost, must be kept well drained, and berms should be built at the toe of slopes of embankments and up to two feet above high water level at locations where total drainage is not practicable. Snowfall is not heavy and temperatures are not lower than encountered on the Trans-Continental Lines in Canada; however, winters are longer.

B) From the Limestone River northerly - where permafrost is general - to Churchill, a distance of 160 miles.

Here the railway runs almost due north over the low watershed between the Nelson River and the Churchill River - a vast area of muskeg, with hundreds of shallow lakes and ponds. Opinions had been expressed that it would not be practicable to construct and maintain a railway there.

Surveys and soil tests, during the winter of 1926-27, showed this to be incorrect. As transport was by dog teams, weight of supplies and equipment had to be kept to a minimum. No drill equipment could be carried so it was necessary to actually chop test-holes to ascertain the depth of organic materials overlying the glacial drift. The greatest depth found was 18 feet. Lenses of clear ice up to 6 inches thick were cut through in both the organic and glacial materials.

Further reconnaissance was made during the following summer by a small party walking across country packing the bare essentials. This confirmed observations made under winter conditions that, almost the entire area was under laid with permafrost and that, generally the annual thaw of the "active" layer extended only to a depth of one to two feet from the surface.

At the same time, the mouth of the Churchill River, a natural harbour, was surveyed with respect to development of a harbour.

As results of these surveys were favourable in comparison with alternatives which had been previously studied, it was decided that, the railway should be constructed to Churchill and a seaport be developed there.

Plans were made to commence location surveys early in November, 1927. In this territory, winter is much more favourable for ground surveys and to search for granular materials for trainfill and ballast. Dogs were, as yet, the only practical means of forward transport, also, aerial photographs and photogrammetry had not become available.

Plans were also made for construction to be commenced in January 1928 and to closely follow the location surveys as they advanced. All supplies and materials, required for the following summer, had to be freighted in and distributed along the route. This was done with horses, hired for the winter from prairie farms; it had to be completed and the teams taken out before Spring break-up.

Originally, all bridges were of the pile trestle type and were built during the same winter, in order that materials and equipment could be hauled from site to site over the frozen ground. Piles were driven into the permafrost with the aid of stream jets. It looked odd to see bridges standing alone with no approach embankments but this avoided delays to grading and tracklaying.

To ensure that permafrost would remain in this condition and future maintenance of track and structures would be the minimum, it was decided to design the railway roadway to be almost entirely embankment and make as few cuttings as practicable.

Initial grading was restricted to minor levelling of the surface mosses etc., just sufficient to permit track to be laid.

Three good deposits of sand and gravel - eskers or beach ridges - were located, the farthest from the line was six miles. Tracks were laid into these pits and power shovels, cars and fuels etc., were placed in them before Spring break-up. Then granular materials were excavated, loaded, distributed and tracks were progressively lifted, commencing from each pit to the main line, then each way along it, until connected with work being carried on from pits to the south and north. When the lifts from the respective pits were completed, the main track was on a roadbed almost entirely embankment, composed of sand and gravel.

As stated above, grading prior to tracklaying was restricted to minor levelling of surface mosses and ditches excavated only where it was essential to drain shallow lakes and ponds; the final roadbed was on undisturbed permafrost. The covering of sand and gravel embankments does not appear to have affected the permafrost below, and may have even assisted in perpetuating this condition.

Track was laid into Churchill 29th March 1929; so this most northerly section of railway has been in operation for 30 years.

Maintenance of track is comparatively light; there is very little settlement, very few weeds grow, life of ties and timber structures is considerably longer than in southern areas and some of the original untreated ties are still in service. As renewals are required, treated ties are being installed and it is expected they will have a lift of fifty years.

Although high winds prevail on the tundra, very few snow drifts accumulate on the track; as the rails are 3 to 4 feet above the general ground elevation, winds clear them. Operation of a snowplow is seldom necessary.

One exception to the good features of maintaining a railway on the tundra is that piles in timber trestles are subject to a very irregular heaving. Piles in the same bent are sometimes affected very differently; one pile may remain in position but the one next to it may heave considerably. Another feature is, piles which heave may remain up or may settle again.

Care in pre-steaming holes, about the diameter of the piles, to thaw just sufficient permafrost to permit the piles to be driven - butt down - will reduce the possibility of heaving; if it does occur it is not too serious a situation as timber bridge decks can be surfaced periodically by cutting heaved piles, and/or, by shimming to take up settlement.

The axiom we follow is - (i) wherever practicable avoid excavations in permafrost, (ii) do not disturb the natural surface insulation and, (iii) endeavor to retain the permafrost condition.

Where topography prevents this axiom to be followed, difficult construction situations may arise. Some of these are best illustrated by reference to the more recent construction of the following branch lines from the main Churchill Railway.

Sherridon to Lynn Lake, 144 Miles, 1952-1953

This line, constructed to serve the mining community of Lynn Lake, was built through rolling country of the Canadian Shield, between latitude 55° and 57°. Some hills had heavy overburdens of varved clay and/or organic materials, within "islands of permafrost," where the railway gradients necessitated making excavations.

These cuts in permafrost were mostly excavated in layers by bulldozers, as materials progressively thawed after exposure. However, where time was

limited, explosives were necessary to break up some of the deeper cuts, for materials to be handled with power shovels. All cuts in permafrost were excavated to additional depth - 2 feet to 3 feet below sub-grade - to permit more ballast to be used in such locations and finally the sides were widened to very flat slopes.

Sipiwesk to Thompson, 31 Miles, 1957

This branch line, located between latitude 55°25' and 55°45', turns off from the Hudson Bay Railway at Mile 200 and runs north-westerly through territory similar to the Lynn Lake Line. Although it is within the Canadian Shield, there is generally heavy overburden. It is well timbered with spruce, aspen, jackpine, birch and willows.

Permafrost was encountered in a number of cuts, through varved clay and bands of ice. Very flat side slopes were necessary to prevent materials from sliding and obstructing drainage ditches in the bottom of these cuts. As time was limited to meet a deadline for track to be laid to a new mine site, these cuts were only excavated to a depth of one foot below normal sub-grade.

The following Spring, 1958, as the permafrost thawed, clay worked up through the ballast and caused serious track problems. These were not overcome until sections were removed to permit excavations to be deepened to at least three feet below sub-grade. These excavations were then back-filled with granular materials, drainage ditches improved, and the track relaid. There is now no difficulty in maintaining a reasonably good surface.

At one location where it was necessary to make a cutting through organic materials, in permafrost condition, additional time was taken to excavate to a depth of five feet below sub-grade and backfill immediately with comparatively dry clay - granular materials were not available at the time. When track was laid, it was given a normal lift of ballast and it remained in surface.

This branch line was constructed to serve a property being developed by the International Nickel Company. This project included a major mining and refining plant and relative townsite. "Islands" of permafrost presented serious foundation problems in connection with some buildings and services in the townsite. Fortunately there are extensive ridges of sand and gravel, within five miles, available for back-filling excavations made in permafrost below sub-grade. Excavations to solid rock were made for foundations of all heavy plant structures.

To provide power for this project the Manitoba Hydro-Electric Board is constructing the Kelsey Generating Station, on the Nelson River, 50 miles north of Thompson. Extensive and deep permafrost was encountered during excavation of the spillway, this caused difficulties in removing such a large volume of frozen layers of clay, silt and ice, which when exposed, became almost fluid.

Optic Lake to Chisel Lake, 52 Miles, 1958-1959

This line, currently under construction to serve a new mine being developed by the Hudson Bay Mining and Smelting Company, runs in a general east-west direction between latitude 54°40' and 54°50'. General characteristics are low hills of exposed rock with muskegs between, excepting at the easterly end, where there is considerable overburden of glacial drift.

This may be considered as the extreme southerly limit of permafrost in Northern Manitoba, only a few small isolated patches are being encountered where heavy surface mosses cover poorly drained organic materials.

CONCLUSION

Based on observations during a period of nearly fifty years, in railway location, construction and maintenance in territory where permafrost occurs, the following general recommendations are submitted:

1) Before ground surveys are commenced to establish a location for a proposed railway, air photos should be analyzed and interpreted to indicate where permafrost may occur and to what extent and to locate possible deposits of granular materials.

2) Air photographs and photogrammetry should also be used to aid in selection of the best route. Careful consideration of drainage features is particularly important.

3) In territory where permafrost occurs in comparatively small isolated areas - "Islands" - such areas should be avoided to the extent practicable in relation to distance and ruling gradients etc.

4) Permafrost on a side-hill slope can become very troublesome.

5) Where it is not practicable to avoid excavating in permafrost for construction of a railway roadway, excavations should be extended to a depth of 3 feet to 5 feet below sub-grade, then backfilled immediately with comparatively dry materials preferably granular if available within reasonable hauling distance.

6) Sides of cuts in permafrost should be excavated to very flat slopes, as a precaution against materials sliding and obstructing drainage ditches.

7) Drainage is always important, this especially so in areas of permafrost. Off-take ditches should be excavated and well maintained to prevent accumulation of water against side-slopes of embankments, particularly where they are built on permafrost, as water in contact with it will cause thawing.

8) Special care is necessary in selecting sites for bridges and culverts and type of structure with respect to site conditions. Whereas first class foundations are essential for rigid type structures, such as steel bridges on concrete sub-structures, it is not so serious if some distortion occurs in a timber trestle or a cedar box culvert; they will remain functional.

9) Cross section area of culverts should be comparatively large to facilitate cleaning in event of an accumulation of ice.

10) In areas where permafrost is general and there are no great changes in topography, such as the tundras, design the location to obtain a roadway to be entirely embankment, if practicable to do so.

11) Relative telegraph and telephone lines should be erected on tripods of cedar poles, set on the surface.

Excavations should be restricted to off-take ditches and minor levelling on the railway right-of-way just sufficient to permit track to be laid. Then the track should be lifted by trainfill, to build up embankments on undisturbed permafrost.

Finally, when constructing where permafrost is general, preserve this condition, build on it, do not excavate into it, except for essential drainage to prevent accumulation of water and relative thawing.

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LINEARLY VARIABLE LOAD DISTRIBUTION ON A RECTANGULAR FOUNDATION

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SYNOPSIS

The vertical stresses and surface displacements caused by a rectangular linearly variable load distribution applied at the level surface of an idealized body are given in terms of equations and charts. An equation is also presented for conveniently estimating the angle of tilt of a rigid plate loaded by a linearly variable load distribution.

INTRODUCTION

In problems involving computations of stresses and displacements caused by non uniform load distributions, it is expected that solutions for linearly variable load distributions are likely to be more useful than the readily available solutions for uniform load distributions. Furthermore, equations giving the angle of tilt of a rigid plate are likely to be of use in connection with rigid structures with layout requirements resulting in eccentric loading, retaining gravity structures, stack-like structures subjected to long term action of prevailing winds, and others.

Derivation of general equations of stresses and displacements for a rectangular linearly variable load distribution must be based on simplifying assumptions rendering feasible the mathematical treatment of the problem. In the following derivations it is assumed that the load distribution is applied on the level surface of a semi-infinite, homogeneous, elastic, and isotropic body. Some consideration of anisotropy is given to the derivation of equations of stress variation with depth. In deriving equations for stresses and surface displacements it is assumed that the load distribution is applied on the body

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1. Soils and Foundations Engr., The Frank E. Basil Co., Inc., Athens, Greece.

through a completely flexible boundary. In deriving equations for tilt it is assumed that the load distribution is applied on a completely rigid boundary.

Computation of Stresses

Fig. 1 shows a rectangle of sides a and b loaded with a load distribution linearly variable in a direction parallel to side a. The coordinate axes Ox and Oy along the sides a and b of the rectangle, respectively, are also shown in Fig. 1. Equations giving the variation of vertical stress with depth underneath any point A of coordinates x and y will be derived.

Since the body is assumed to be elastic the stresses caused by the assumed load distribution are equal to the sum of stresses caused by the uniform load distribution (u) and the variable load distribution (v) shown in Fig. 1. Consequently

$$\sigma_z = \sigma_{zu} + \sigma_{zv} \tag{1}$$

where σ_z is the total stress, σ_{zu} is the vertical stress caused by the uniform load distribution (u), and σ_{zv} is the vertical stress caused by the variable load distribution (v). Quantity σ_{zu} can be computed by means of known equations or charts (e.g. page 255 of ref (3)). Equations giving σ_{zv} will be derived in the following paragraphs by integrating the equation of stress caused by a perpendicular point surface force.

In the case of an isotropic as well as homogeneous semi-infinite elastic body the stress caused by a perpendicular point surface force is, according to Boussinesq, equal to

$$\sigma_{zp} = \frac{P}{2\pi} \frac{3z^3}{(r^2+z^2)^{5/2}} \tag{2}$$

where σ_{zp} is the stress caused by the point load, P is the surface force, z is the depth, and r is the distance between the point where the stress exists and the line of force P. If AX and AY are two coordinate axes parallel to Ox and Oy respectively (Fig. 1),

$$r^2 = X^2 + Y^2 \qquad P = kX\,dX\,dY$$

where X, Y are coordinates of any point inside the loaded rectangle with respect to AX and AY axes, k is the rate of change of load distribution in a direction parallel to Ox and OX, and dX, dY are the sides of an elementary loaded area, located around point X, Y. The contribution $d\sigma_{zv}$ to the vertical stress σ_{zv} of the elementary loaded area dXdY is equal to:

$$d\sigma_{zv} = \frac{kX\,dX\,dY}{2\pi} \frac{3z^3}{(X^2+Y^2+z^2)^{5/2}}$$

and:

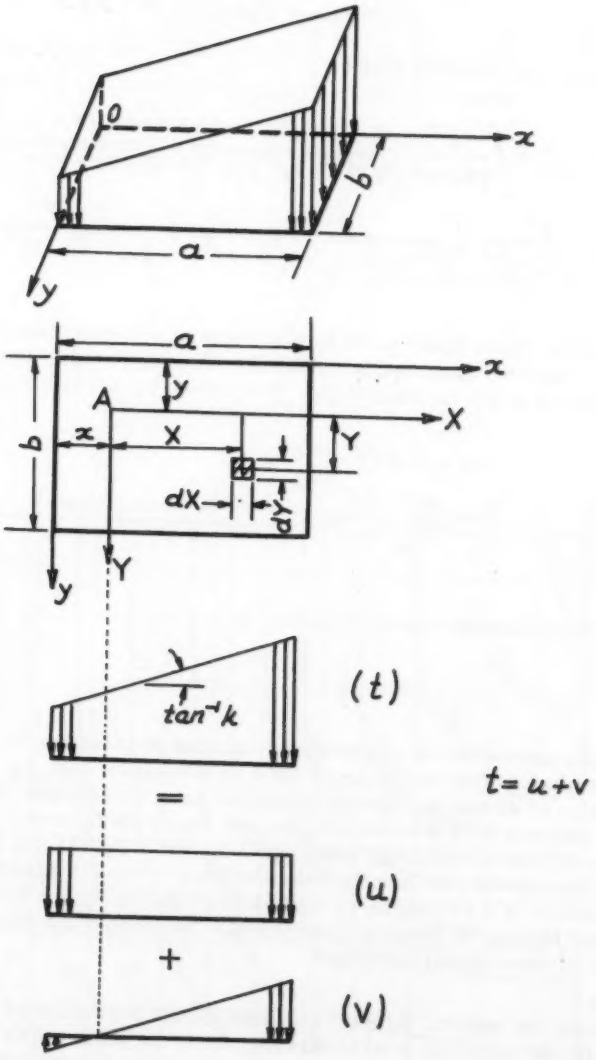

FIGURE 1

$$\sigma_{zv} = \frac{3}{2\pi} k z^3 \int_{Y=-y}^{Y=b-y} \int_{X=-x}^{X=a-x} \frac{X\,dX\,dY}{(X^2+Y^2+z^2)^{5/2}}$$

Integration of this equation gives:

$$\sigma_{zv} = \frac{1}{2\pi} k \left[\frac{z^3}{z^2+x^2} \left\{ \frac{b-y}{\sqrt{z^2+x^2+(b-y)^2}} + \frac{y}{\sqrt{z^2+x^2+y^2}} \right\} - \frac{z^3}{z^2+(a-x)^2} \left\{ \frac{b-y}{\sqrt{z^2+(b-y)^2+(a-x)^2}} + \frac{y}{\sqrt{z^2+(a-x)^2+y^2}} \right\} \right] \quad (3)$$

The use of Eq. (3) is likely to be cumbersome in problems where many stress values are desired. In order to render Eq. (3) more readily usable the special case of x=y=o is considered. For x=y=o Eq. (3) becomes:

$$\sigma_z = k z N \quad (4)$$

$$N = \frac{m}{2\pi} \left(\frac{1}{\sqrt{1+m^2}} - \frac{1}{1+n^2} \frac{1}{\sqrt{1+m^2+n^2}} \right) \quad (5)$$

where N is a dimensionless parameter and

$$m = \frac{b}{z} \qquad n = \frac{a}{z}$$

For easier application N is given as a function of m and n in Fig. (2) in the form of a chart. This chart can be used in connection with Eq. (3) for the computation of stresses underneath the corner of a rectangle loaded with a linearly variable load distribution, the unit load directly above the point at which the stress is computed being equal to zero. Eq. (3) and the chart of Fig. (2) can also be used for the computation of stresses underneath a point in the interior of a rectangle, by considering separately the four smaller rectangles formed by drawing lines through the point in question parallel to the sides of the original rectangle.

Example 1

Compute the vertical stress 7 feet underneath the corner of a rectangle 10 × 15 ft; the unit load is zero directly above the point where the stress is desired and varies linearly along the 10 ft dimension at the rate of 250 lbs/cu ft.

In this case m = 15:7 = 2.14, n = 10:7 = 1.43, N = 0.11, k = 250 lbs/cu ft, z = 7 ft, σ_{zv} = 250 × 7 × 0.11 = 193 lbs/sq ft.

Example 2

Compute the vertical stress 18 feet underneath point B in the rectangle shown in Fig. 3.

Stress σ_{zu} (Eq. (1)), caused by the uniform load distribution ℓ shown in Fig. 3 (u), can be computed by means of known equations or charts. Stress σ_{zv} can be computed using Eq. (4).

k = 80 lbs/cu ft, z = 18 ft and

for rectangle I, m = 5:18 = 0.28, n = 7:18 = 0.39, N = 0.009

for rectangle II, m = 5:18 = 0.28, n = 23:18 = 1.28, N = 0.033

for rectangle III, m = 15:18 = 0.83, n = 23:18 = 1.28, N = 0.070

for rectangle IV, m = 15:18 = 0.83, n = 7.18 = 0.39, N = 0.017

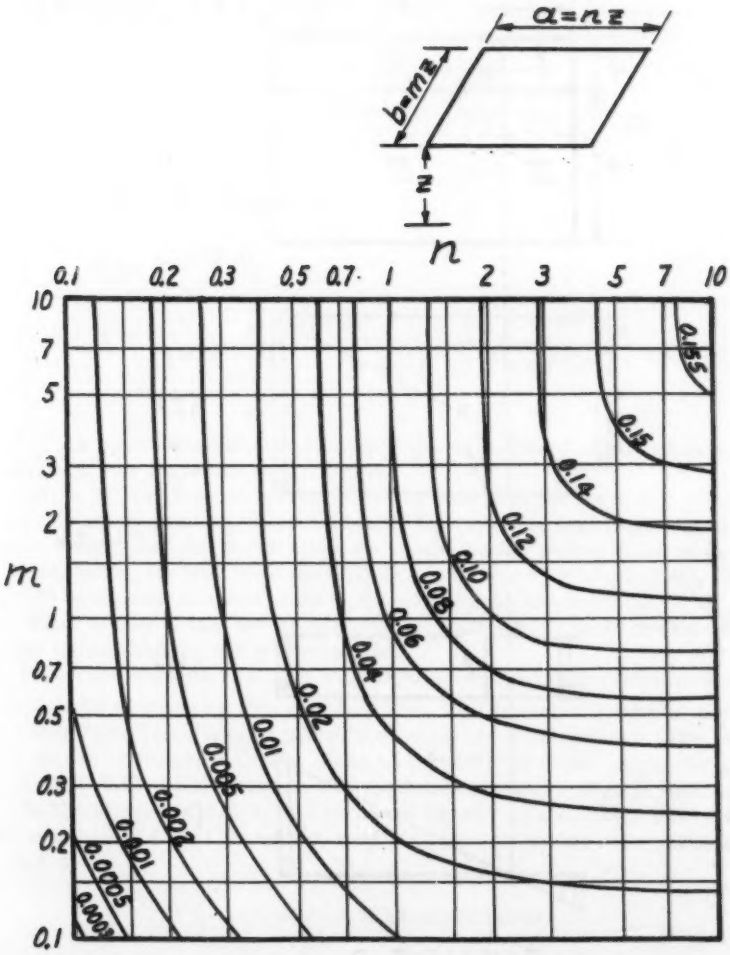

FIG. 2 VARIATION OF N WITH m AND n

The contributions of rectangles II and III to $\sigma_{\pm V}$ are positive whereas the contributions of rectangles I and IV are negative; therefore:

Summation of N = 0.033 + 0.070 - 0.009 - 0.017 = 0.077

and $\sigma_{\pm V}$ = 80 × 18 × 0.077 = 111 lbs/sq ft.

Eqs (3) and (4) are based on Eq. (2) which, in turn, is based on the assumption of an isotropic body. In cases, however, where a stratified body appears to be a more reasonable assumption in estimating stresses, stress computations should probably be based on Westergaard's equation:

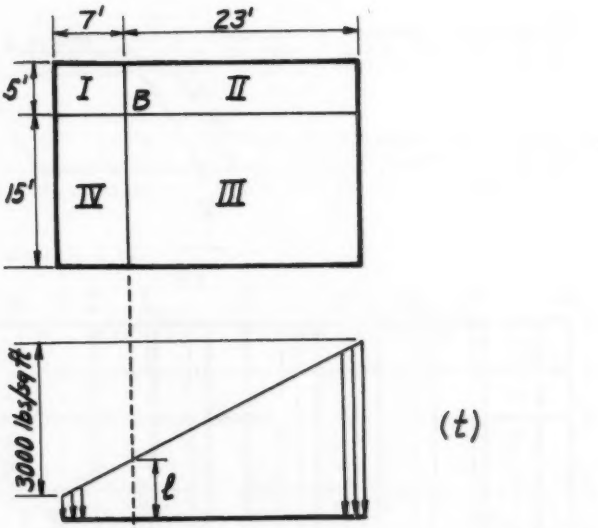

FIGURE 3

$$\sigma_{zP} = \frac{P}{\pi} \frac{1}{z^2\left[1+2\left(\frac{r}{z}\right)^2\right]^{3/2}} \tag{2A}$$

Eq. (2A) is for the case of a body prevented as a whole from undergoing lateral strain and for a value of Poisson's ratio equal to zero. Integration over the loaded rectangle of the contribution $d\sigma_{zv}$ to the vertical stress σ_{zv} of an elementary loaded area dXdY, gives

$$\sigma_{zP} = \frac{1}{2\sqrt{2}\,\pi} k z \log\left[\frac{\sqrt{(a-x)^2+y^2+\frac{1}{2}z^2}-y}{\sqrt{(a-x)^2+(b-y)^2+\frac{1}{2}z^2}+b-y} \cdot \frac{\sqrt{x^2+(b-y)^2+\frac{1}{2}z^2}+b-y}{\sqrt{x^2+y^2+\frac{1}{2}z^2}-y}\right] \tag{3A}$$

For x = y = o Eq. (3A) becomes:

$$\sigma_{zv} = k z N' \tag{4A}$$

$$N' = \frac{1}{2\sqrt{2}\,\pi} \log \frac{\sqrt{1+2n^2}\left(m+\sqrt{\frac{1}{2}+m^2}\right)}{m+\sqrt{\frac{1}{2}+m^2+n^2}} \tag{5A}$$

where N' is a dimensionless parameter; the logarithms indicated in Eqs. (3A) and (5A) are Naperian logarithms.

In Fig. 4, N' is given as a function of m and n in the form of a chart. This chart can be used in connection with Eq. (3A), for the computation of stresses underneath the corner of a rectangle loaded with a linearly variable load distribution, the unit load directly above the point at which the stress is computed being equal to zero. As in the case of the chart of Fig. 2, the use of the chart of Fig. 4 can be extended to the computation of stresses underneath points in the interior of rectangles.

In Fig. 5 the variation of σ_{zv} with depth underneath the corners of a square (a = b) is given for the cases of Eqs. (4) (Boussinesq type of stress distribution) and (4A) (Westergaard type of stress distribution). The variation of stress underneath C (Fig. 5) was computed by direct application of Eqs. (4) and (4A). The variation of stress underneath D (Fig. 5) was computed as the algebraic sum of the stresses caused by a uniform load distribution equal to $\Delta\ell$ and a negative load distribution having zero value at D and $-\Delta\ell$ at C.

Computation of Displacements

The vertical displacement of a point A (Fig. 1) on the level surface of a semi-infinite, homogeneous, elastic, and isotropic body, caused by a

rectangular linearly variable load distribution applied at the surface, can be expressed in terms of equations and charts. Since the body is assumed to be elastic the displacement of point A (Fig. 1) caused by the assumed load distribution is equal to the sum of displacements caused by the uniform load distribution (u) and the variable load distribution (v), as shown in Fig. 1.

$$W = W_u + W_v \tag{6}$$

where w is the total vertical displacement of point A, w_u is the vertical displacement of point A caused by the uniform load distribution (u) (Fig. 1) and

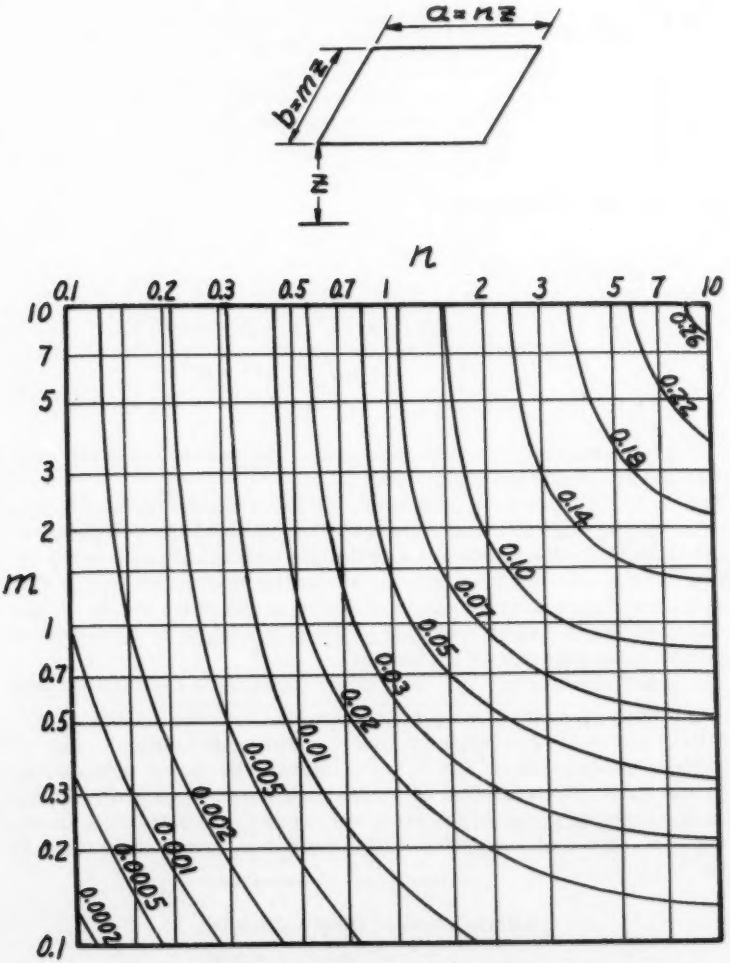

FIG. 4. VARIATION OF N' WITH m AND n

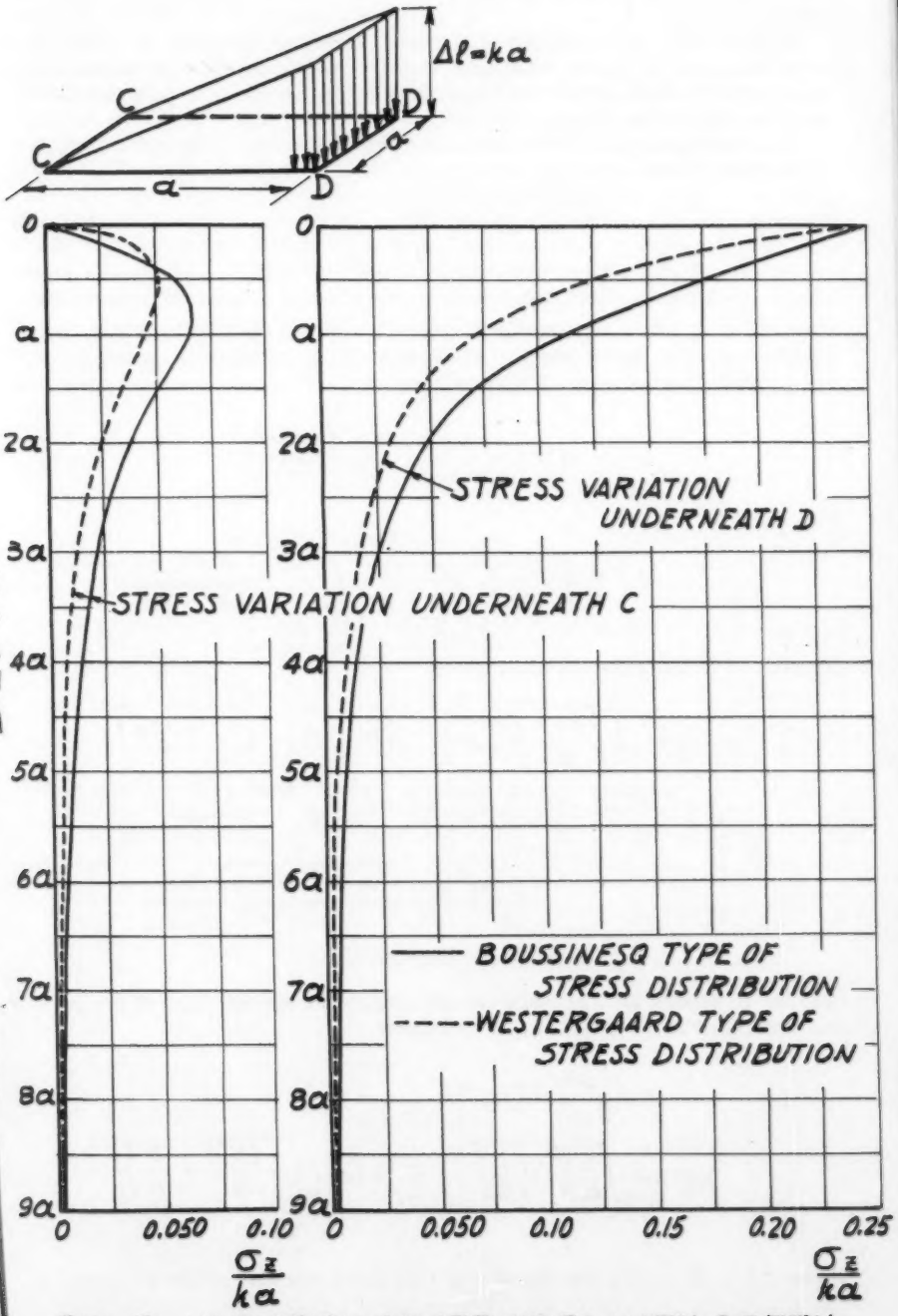

FIG. 5 VARIATION OF STRESSES WITH DEPTH

w_v is the vertical displacement of point A caused by the variable load distribution (v) (Fig. 1).

Quantity w_u can be computed by means of known equations or charts (e.g. page 383 of ref (4)). An equation giving w_v can be derived by integrating the equation of vertical displacement at the surface caused by a perpendicular point surface force.

The vertical displacement at the surface caused by a point force is given by (section 123 of ref. (5)).

$$w_p = \frac{P(1-\nu^2)}{\pi E r} \tag{7}$$

where w_p is the vertical displacement of a point at a distance r from the point force P and ν is Poisson's ratio. From Eq. (7) it follows that the contribution dw_v to the vertical displacement w_v of the elementary loaded area dXdY, located around X, Y is equal to:

$$dw_v = \frac{kX\,dX\,dY\,(1-\nu^2)}{\pi E \sqrt{X^2+Y^2}}$$

and

$$w_v = \frac{1}{\pi}\,\frac{1-\nu^2}{E}\,k \int_{Y=-y}^{Y=b-y} \int_{X=-x}^{X=a-x} \frac{X\,dX\,dY}{\sqrt{X^2+Y^2}}$$

Integration of the above equation gives:

$$w_v = \frac{1}{\pi}\,\frac{1-\nu^2}{E}\,k \left[\frac{b-y}{2}\left(\sqrt{(a-x)^2+(b-y)^2} - \sqrt{x^2+(b-y)^2}\right) - \frac{y}{2}\left(\sqrt{x^2+y^2} - \sqrt{(a-x)^2+y^2}\right) - \frac{x^2}{2}\log\frac{\sqrt{x^2+(b-y)^2}+b-y}{\sqrt{x^2+y^2}-y} + \frac{(a-x)^2}{2}\log\frac{\sqrt{(a-x)^2+(b-y)^2}+b-y}{\sqrt{(a-x)^2+y^2}-y} \right] \tag{8}$$

In order to render Eq. (8) more readily usable the special case of x = y = o is considered. For x = y = o Eq. (8) becomes:

$$w_v = \frac{1-\nu^2}{E}(\Delta\ell)\,aM \tag{9}$$

$$M = \frac{1}{2\pi}\left[\frac{b}{a}\left(\sqrt{1+\left(\frac{b}{a}\right)^2} - \frac{b}{a}\right) + \log\left(\sqrt{1+\left(\frac{b}{a}\right)^2} + \frac{b}{a}\right)\right] \tag{10}$$

where $\Delta\ell$ = ak = total change of unit load from one side of the rectangle to

the other, and M is a dimensionless parameter. All logarithms shown in Eqs. (8) and (9) are Naperian.

In order to simplify the use of Eq. (9) the variation of M with the ratio a:b is given in Fig. 6. Insertion of the value of M in Eq. (9) will give the vertical displacement at the corners of zero load C (Fig. 6) for the linearly variable load distribution. In Fig. 6 the value of another parameter M' is also given; this is the value that must be inserted in Eq. (9) instead of M, in order to obtain the settlement at the corners of maximum load D (Fig. 6). The variation of M' with a:b was computed as the algebraic sum of the displacements caused by a uniform load distribution equal to $\Delta\ell$ and a negative linearly variable load distribution having zero value at D and $-\Delta\ell$ at C. The use of Fig. 6 and Eq. (9) in computing settlements is illustrated in the following example.

Example 3

Compute the vertical displacement for the loading given in example 1 at one of the corners of zero load; E = 100,000 lbs/sq ft, ν = 0.4. In this case

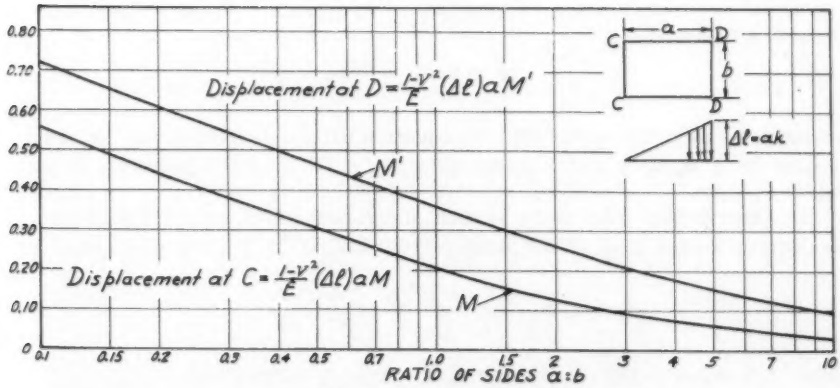

FIG. 6 VARIATION OF M WITH a:b

$\Delta \ell = 250 \times 10 = 2500$ lbs/sq ft, a = 10 ft, a:b = 10:15 = 0.67, M = 0.26, and from Eq. (9):

$$w_v = \frac{1 - 0.16}{100{,}000} \times 2000 \times 10 \times 0.26 = 0.044 \text{ ft}$$

The use of Eq. (9) and Fig. 6 can be extended to the computation of vertical displacements in the interior of rectangles by a method similar to the method illustrated in example 2.

Tilt of a Rigid Plate

The tilt of a rigid plate resting on a semi-infinite, homogeneous, elastic, and isotropic body will be estimated for the case of a linearly variable load distribution. The tilt depends on the shape of the loaded rectangle and the change in unit load from end to end and is independent of the uniform component of the load distribution. The absolute value of the unit load at the two ends of the rectangle does not affect the tilt and can be assumed, for the sake of convenience, that it is equal to zero at one end and equal to a maximum value $\Delta \ell$ at the other end (Fig. 7). Another assumption made in estimating the angle of tilt is that rigid plate displacements are equal to the average displacements produced by a flexible load distribution, the average values being compatible with the necessary condition of a plane bottom. The displacement w at any point A of coordinates x, y (Fig. 7) in the rectangle is, according to Eq. (6):

$$w = w_u + w_v \tag{6}$$

Quantity w_v is given by Eq. (8). An equation will be derived giving w_u as a function of a, b, x, y and k by integrating Eq. (7) over the rectangle of loading, as follows.

The contribution dw_u to the vertical displacement w_u of point A of the elementary loaded area dXdY located around point X, Y (Fig. 7) is equal to:

$$dw_u = \frac{1}{\pi} \frac{1-\nu^2}{E} \ell \frac{dX\,dY}{\sqrt{X^2+Y^2}}$$

where ℓ is equal to the unit load at A ($\ell = kx$). It follows that

$$w_u = \frac{1}{\pi} \frac{1-\nu^2}{E} \ell \int_{Y=-y}^{Y=b-y} \int_{X=-x}^{X=a-x} \frac{dX\,dY}{\sqrt{X^2+Y^2}}$$

Integration of the above equation and insertion of kx in the place of ℓ gives:

$$w_u = \frac{1}{\pi} \frac{1-\nu^2}{E} k \left[(a-x)\, x \log \frac{\sqrt{(a-x)^2+(b-y)^2} + b-y}{\sqrt{(a-x)^2+y^2} - y} + \right.$$

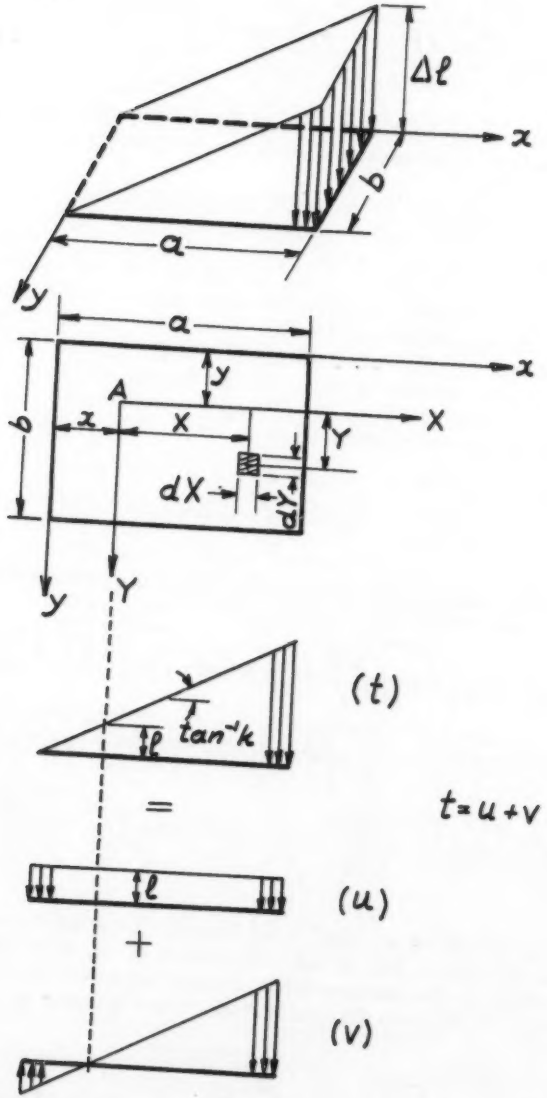

FIGURE 7

$$x^2 \log \frac{\sqrt{x^2+(b-y)^2}+b-y}{\sqrt{x^2+y^2}-y} + x\,y \log \frac{\sqrt{(a-x)^2+y^2}+a-x}{\sqrt{x^2+y^2}-x} +$$

$$x(b-y) \log \frac{\sqrt{(a-x)^2+(b-y)^2}+a-x}{\sqrt{x^2+(b-y)^2}-x} \Bigg] \qquad (11)$$

Eq. (6), in connection with Eqs. (8) and (11), gives the settlement of any point on the surface of the idealized body for the flexible linearly variable loading shown in Fig. 7.

Substitution of x by (a - x) in Eq. (8) results in reversal of sign, i.e., at points symmetrical about line x = 0.5a, w_v has values equal and opposite. Quantity w_v is positive from x = o to x = 0.5a and negative from x = 0.5a to x = a, being equal to zero for x = 0.5a. It follows that the contribution of w_v to the tilt of the rigid plate is a rotation about axis x = 0.5a the angle of tilt causing a volume displacement on either side of the line x = 0.5a equal to the volume created by w_v over the first half (from x = o to x = 0.5a) of the rectangle. The volume created by w_v over half the rectangle can be computed by integrating the expression of Eq. (8) and the angle of tilt can be computed by multiplying the result of integration by $8{:}a^2b$. The result of these operations is

$$T_v = \frac{1-\nu^2}{E}(\Delta \ell)\, I_v$$

where T_v is the contribution of w_v to the tilt of the plate in radians and

$$I_v = \frac{2}{3\pi}\left[4 \log\left(\sqrt{1+\left(\frac{b}{a}\right)^2}+\frac{b}{a}\right) - \log\left(\sqrt{1+\left(\frac{2b}{a}\right)^2}+\frac{2b}{a}\right) + \frac{7}{4}\frac{a}{b} - \right.$$

$$\frac{a}{b}\left(\sqrt{1+\left(\frac{b}{a}\right)^2}-\sqrt{\frac{1}{4}+\left(\frac{b}{a}\right)^2}\right)\left\{\frac{5}{2}+2\sqrt{1+\left(\frac{b}{a}\right)^2}\sqrt{\frac{1}{4}+\left(\frac{b}{a}\right)^2}-\left(\frac{b}{a}\right)^2\right\} -$$

$$\left. 2\left(\frac{b}{a}\right)^3 \log\left(\sqrt{\frac{1}{4}+\left(\frac{b}{a}\right)^2}+\frac{1}{2}\right) + \left(\frac{b}{a}\right)^3 \log \frac{b}{a}\left(\sqrt{1+\left(\frac{b}{a}\right)^2}+1\right)\right] \qquad (13)$$

As the expression of w_u (Eq. (11)) does not show the type of symmetry observed in the case of Eq. (8) the contribution of w_u to the angle of tilt can not be computed by the same method. Integration of the expression of Eq. (11) with respect to y between the limits O and b and division by b gives:

$$w_{uy} = \frac{1}{\pi}\frac{1-\nu^2}{E}\frac{k}{b}\left[(a-x)\,x\,b \log \frac{\sqrt{\frac{(a-x)^2}{b^2}+1}+1}{\sqrt{\frac{(a-x)^2}{b^2}+1}-1} + x^2 b \log \frac{\sqrt{\left(\frac{\cdot}{b}\right)^2+1}+1}{\sqrt{\left(\frac{x}{b}\right)^2+1}-1}\right.$$

$$+ b^2 x \log \frac{\sqrt{\frac{(a-x)^2}{b^2}+1} + \frac{a-x}{b}}{\sqrt{\left(\frac{x}{b}\right)^2+1} - \frac{x}{b}} + a^2 x + 2x^3 - 2ax^2 -$$

$$(a-x)\, x \sqrt{(a-x)^2 + b^2} - x^2 \sqrt{x^2 + b^2} \Big] \qquad (14)$$

where w_{uy} is the average value of w_u over the y dimension. The contribution of w_u to the angle of tilt of the rigid plate was computed by plotting the value of w_{uy} as a function of x from O to a and drawing the average line for several values of the ratio a:b. In drawing the "average line" the positive and negative areas between the "average line" and the actual curves as well as the moments of these areas were balanced. The total tilt of the plate is equal to:

$$T = T_u - T_v = \frac{1-\nu^2}{E} (\Delta \ell)\, I \qquad (15)$$

where T is the total angle of tilt in radians, T_u is the contribution of w_u to the tilt, also in radians, and I is a dimensionless parameter depending only on the ratio a:b. The variation of I with the ratio a:b is given in Fig. 8.

Eq. (15) and the graph of Fig. 8 are intended for quick estimates of angles of tilt of rigid structures resting on a body that may, without great error, be considered semi-infinite, homogeneous, elastic, and isotropic.

Example 4

Compute the angle of tilt for a rigid plate of the dimensions and loading of example 3.

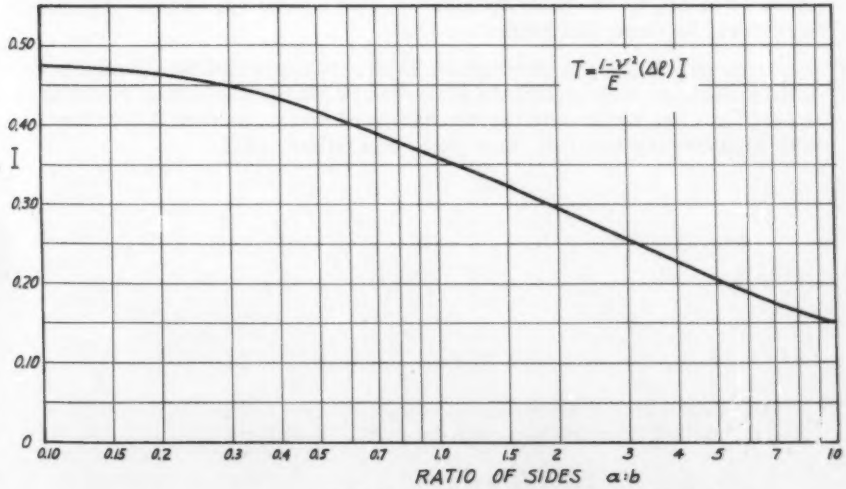

FIG. 8 VARIATION OF I WITH a:b

For the value of a:b = 0.67, I = 0.40 and

$$T = \frac{1 - 0.16}{100,000} \times 2500 \times 0.40 = 0.0084 \text{ radians (= slope of 0.84\%)}$$

CONCLUSIONS

Equations and charts giving vertical stresses and surface displacements in the case of a linearly variable load distribution acting on a rectangular area at the surface of an idealized body are presented in this paper. Stresses and surface displacements for the case of the loading acting on a flexible boundary can be conveniently computed partly by means of Eqs. (4), (4A), (9), used in connection with the graphs of Figs. 2, 4, 6, respectively, and partly by methods already known. Angles of tilt of rigid plates can be conveniently estimated by means of Eq. (15) used in connection with the graph of Fig. 8.

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Note: Paper 2325 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, SM 6, December, 1959.

*There will be no closure.

GEOTECHNICAL PROPERTIES OF GLACIAL LAKE CLAYS[a]

Closure by T. H. Wu

T. H. WU,[1] M. ASCE.—The relationship between c/p and the plasticity index is empirical. The writer however, cannot agree completely with Mr. Kenney's views regarding the value of c/p. In the first place, the magnitude of the pore-water pressure at failure in the unconfined compression test is always very small and its value is usually negative. Furthermore, the material fails at normal stresses much smaller than its preconsolidation pressure. Therefore, Eq. (5), based on the assumption that c' = 0, is not valid for the stress conditions existing in the unconfined compression test.

The writer accepts Mr. Kenny's equation for the vane test, but he is not certain that Mr. Kenney's conclusion regarding the relationship between the c/p and the plasticity index is justified. It is true that the value of c/p depends more on the value of A than on that of ϕ'. However, it is very difficult to predict A. Outside of the knowledge that high values of A have been measured in quick clays, little is known. The data presented by Mr. Kenney do not indicate any consistent values of A for clays in the various sensitivity groups. In fact, since A is dependent to a large extent on the compressibility and expansibility, it may bear some relationship to an index property such as the plasticity index. Although the writer is by no means an advocate of the empirical approach, he is forced to the opinion that the present state of knowledge is insufficient for a thorough analysis of the c/p ratio.

Mr. Kenney's comments on the topic of varved clays is instructive. The writer also realizes that varved clays consist of layers of very different material. However, he has found it virtually impossible to separate the elements in a study of this type. The thickness of most of the silt layers lies between the range from 1.0 mm to 3.0 mm. There are thinner laminations, some barely visible to the naked eye, as well as one or two layers as thick as 7 mm. The transition from the silt to the clay is gradual as may be expected. The variation in the thickness of the clay layers is even greater. In general layers with a thickness of less than 10 mm. are very uncommon. In all the unconfined compression tests there is no indication of the composite type of failure.

Lastly, Mr. Kenney asked some interesting questions about the geological origin of the till—like clay deposits. On this topic, the writer regrets that he is unable to offer any comments since very little factual information is available. It may be noted however, that G. H. Otto, in his thesis, stated that the soft clays in the Chicago area form continuous strata with till sheets of stiff clay in the surrounding areas. His hypothesis is that the soft clays consist of till deposited under water.

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a. Proc. Paper 1732, August, 1958, by T. H. Wu.

1. Associate Prof., Civ. Eng., Michigan State Univ., East Lansing, Mich.

TRACTIVE RESISTANCE OF COHESIVE SOILS[a]

Discussion by James L. Sherard

JAMES L. SHERARD,[1] M. ASCE.—The author is to be complimented for presenting the results of his very interesting researches on the resistance of soils to the erosive forces of flowing water.

While the work was done for the purpose in aiding in the solution of the problem created by erosion of the walls and bottoms of unlined channels, the instrumentation and method of laboratory testing may be used profitably in future researches to approach another important problem—the study of the erosive resistance of compacted earth dam embankments against seepage forces which develop as a result of leaks ("piping" resistance).

At the present state of our ability to analyze earth dams, the engineer does not consider the "tractive" or "piping" resistance of the compacted impervious core except in a very general and qualitative manner. In current practice, the design of filters which are constructed in earth dams to control piping is governed only by the grain size characteristics of the protected material. However, the writer believes that future developments in this field may be such that much more consideration will be given to the inherent resistance of the soil against progressive erosion of leaks which occasionally develop through earth dams in spite of the most careful construction control. At this future date, laboratory investigations of the type carried out by the author or of some other technique will be necessary to determine the influence of soil properties and compaction method on the "piping" resistance.

This discussion contains some very rough and empirical correlations between earth dam soil properties and "piping" resistance found in an investigation carried out by the writer(1) and a few comments and questions about the author's work.

Earth Dam Piping Resistance

Piping or progressive erosion of leaks which develop under and through earth dams has accounted for a large percentage of the catastrophic types of failures which have occurred. Records of the performance of many dams clearly demonstrate that there is a wide range of resistance to piping in earth dams. Some dams have failed completely as the result of progressive erosion of what was once a slight toe leak. Other dams have withstood large concentrated leaks directly through the embankment for years without any progressive piping.

In the following paragraphs, "resistance to piping" is used to denote resistance to progressive backward erosion once initial condition of leakage

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a. Proc. Paper 2062, June, 1959, by Irving S. Dunn.

1. Woodward-Clyde-Sherard and Associates, Cons. Engrs., New York, N. Y.

is established. A concentrated leak occurring under or through an embankment when the reservoir is filled for the first time indicates only that the water has found a channel. Piping resistance refers to the resistance of the embankment material to the erosive action of the seeping water.

The principal purpose of the writer's study was to scrutinize the characteristics of the earth dam which govern this resistance to piping. The problem was approached by making a careful study of the performance of 31 earth dams in the western part of the United States which developed leakage. Some of these dams failed and some demonstrated resistance to piping in various degrees. In order to have a performance record of sufficient length, only dams over thirty years of age were included in this study.

As a part of the study, good records of the performance of each dam were obtained. Also soil samples were obtained which were determined to be typical or average of the embankment material.

Three general categories of piping resistance were arbitrarily defined and, after an evaluation of the performance record, each dam was assigned to one of the categories. These categories were defined as follows:

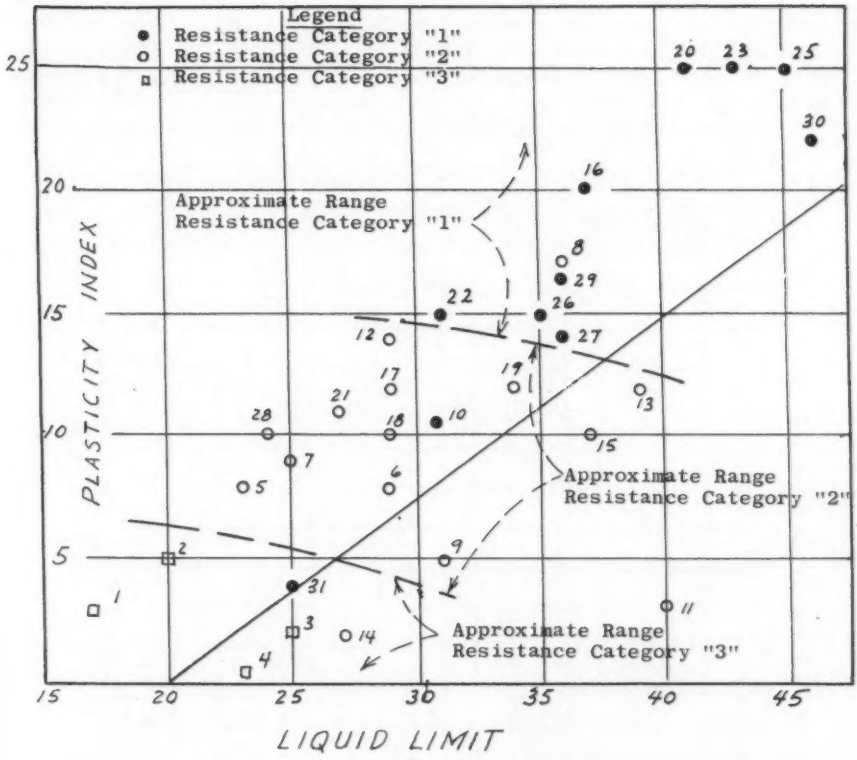
Piping Resistance, Category No. 1.—Greatest resistance to piping. Small and medium, concentrated leaks will heal themselves or continue to flow clearly without an increase in quantity. The embankment may fail completely as a result of slowly progressive piping caused by a large (defined arbitrarily as approximately 1/2 c.f.s. or greater) concentrated leak. This progressive erosion or piping will occur slowly and give plenty of time for remedial action to be taken.

Piping Resistance, Category No. 2.—Intermediate resistance to piping. Safely resists saturation of the lower portions of the downstream slope indefinitely. May fail eventually as result of erosion caused by small concentrated leakage but only after a long period of time. If a large leak through the embankment develops in any manner, piping causes complete failure in a short period of time.

Piping Resistance, Category No. 3.—Least resistance to piping. Usually fails completely within a few years after first reservoir filling if any water finds its way to the unprotected downstream slope. May resist saturation of the downstream toe safely for years but this condition represents a dangerous situation. Any small concentrated leakage on the downstream slope will probably cause failure within a short period of time.

The influence of the soil properties and the embankment construction method on the piping resistance which each dam demonstrated was studied as carefully as the rough nature of the information available permitted. Fig. 1 shows the relationship between the plasticity characteristics of the average embankment material and the piping resistance. As seen in this figure nearly all the dams constructed from soils with a plasticity index in excess of 15 were in the piping resistance Category No. 1 (highest resistance). All the dams which fell in piping resistance Category No. 3 (least resistance) were constructed from soils with plasticity index less than 5. From the data available, it appeared that the plasticity index was the most important parameter governing piping resistance but that the piping resistance increased with increasing liquid limit for a constant plasticity index.

A study of the relationship between piping resistance and gradation of the average embankment soils was not very rewarding. As seen in Fig. 2, the



List of Dams Included in Study and Plotted Above

1. Hatchtown
2. Hebron
3. Frazier
4. Holmes Creek
5. McMillan
6. Magic
7. Long Tom
8. Crane Creek
9. Sinker Creek
10. Jumbo
11. Drum
12. Costilla
13. Eldorado
14. Encono
15. Kunkle
16. Lake Malloya
17. Portneuf
18. Rocky Ford
19. Stanislaus
20. Teller
21. Worster
22. Narraguinnep
23. Bear Gulch
24. Barton
25. Standley Lake
26. Lake Francis
27. Masterson
28. Scofield
29. Fruitgrower's
30. Murphy
31. Sublette

Figure 1

Relationship between piping resistance of earth dams and plasticity indices of average embankment material

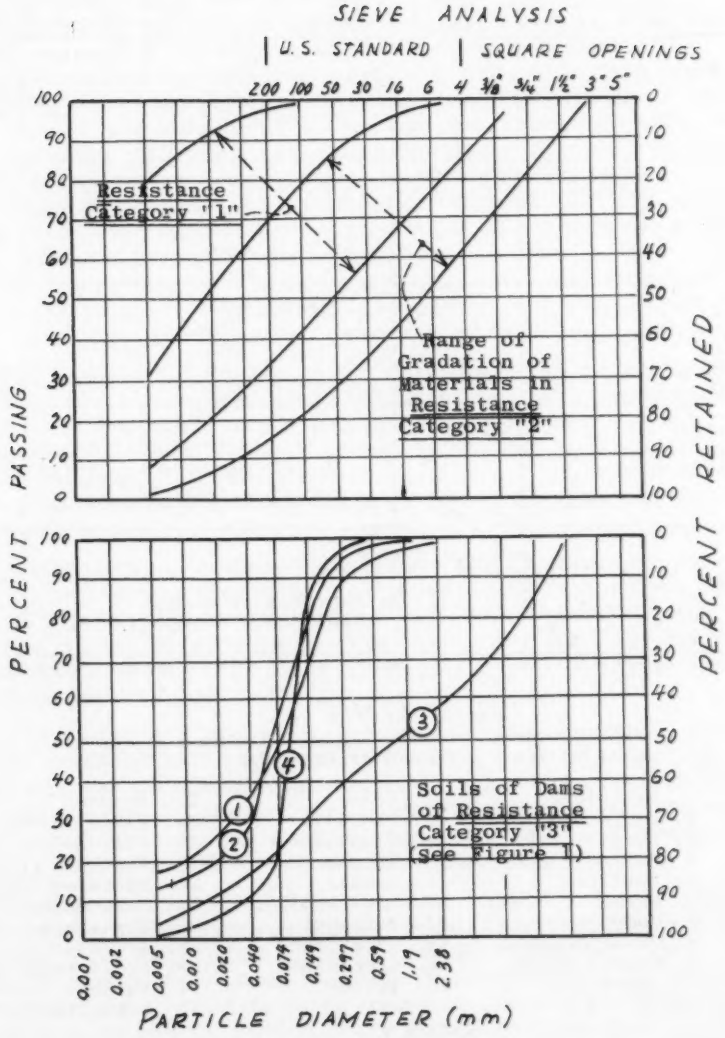

SIEVE ANALYSIS
U.S. STANDARD SQUARE OPENINGS
200 100 50 30 16 6 4 3/8" 3/4" 1 1/2" 3" 5"
PERCENT PASSING
PERCENT RETAINED
Resistance Category "1"
Range of Gradation of Materials in Resistance Category "2"
Soils of Dams of Resistance Category "3" (See Figure 1)
1
2
3
4
0.001 0.002 0.005 0.010 0.020 0.040 0.074 0.149 0.297 0.59 1.19 2.38
PARTICLE DIAMETER (mm)

Figure 2

Relationship between piping resistance of earth dams and gradation of average embankment material

soils in resistance Category No. 1 were finer than the soils in Category No. 2. It is interesting to note that of the four dams whose resistance to piping was such that they were placed in Category No. 3, three were very similar, very uniform and fine sands. Study of the relationship between piping resistance and the method by which the dam was constructed, indicated that the construction was a much less important factor than the soil type. A final summary of the influence of embankment soil type and construction methods on piping resistance as indicated by the study is given in Table 1.

Test Procedures and Interpretation

The test procedures used by the author were particularly interesting. Since it was possible to determine and reproduce the critical head "H" at

TABLE 1

RELATIONSHIP BETWEEN PIPING RESISTANCE OF EARTH DAM EMBANKMENTS AND SOIL TYPES AND CONSTRUCTION METHODS
(In Order Of Decreasing Piping Resistance)

Greatest Piping Resistance Category (1)	1. Plastic Clay, (PI>15), Well compacted. 2. Plastic clay, (PI>15), Poorly compacted.
Intermediate Piping Resistance Category (2)	3. Well-graded material with clay binder, (6<PI<15), Well compacted. 4. Well-graded material with clay binder, (6<PI<15), Poorly compacted. 5. Well-graded, cohesionless material, (6>PI), Well compacted.
Least Piping Resistance Category (3)	6. Well-graded, cohesion-less material, (6>PI), Poorly compacted. 7. Very uniform, fine, cohesionless sand. (6>PI), Well compacted. 8. Very uniform, fine, cohesionless sand, (6>PI), Poorly compacted.

which erosion commenced with the relatively simple set up, it is difficult to imagine how the equipment and procedures could be improved.

On the other hand, there must be a number of other conceivable approaches to the problem of investigating the "erosive" resistance of a compacted soil specimen. It would be of interest if the author would describe briefly what other test setup he experimented with, if any, before arriving at his final procedure. The writer once considered attempting a research program directed at studying "piping" resistance for earth dam material in the laboratory by forcing water through a cylindrical hole in a soil specimen compacted in a Proctor mold and measuring the weight of eroded soil as a function of various parameters of rate and time of water flow. Other investigations have tested the efficacy of filters in earth dams for controlling the seepage through embankment cracks by cutting notches in compacted specimens in more or less standard permeability tests.

One of the principal results of the investigation was the fact that a given soil at a given shear strength (which is a function of the method of sample preparation) demonstrated a definite and reproducible critical "tractive" force or water velocity above which erosion took place. It was further shown as would have been expected that for a given shear strength, the higher the plasticity index, the higher is the critical tractive force. However, the writer has the general feeling that the elaborate correlation made between the soil properties and critical tractive force is perhaps stretching the data obtained a little too far. If the laboratory tests were extended to include a wider range of soil types and compaction efforts (evidently all the samples were relatively soft), another and perhaps stronger correlation might be obtained.

By mixing the soil samples at a water content in excess of the liquid limit and compressing them under pressures of 25 to 300 psf, the samples tested must have had relatively low densities and high water contents. It would be interesting to know what correlations the author found between the density and water contents of the soil specimens and tractive forces.

The device for determining the horizontal force exerted by the flowing water on the soil specimen was particularly ingenious. Since the critical shear stresses at which erosion occurred in the specimens tested were only of the order of magnitude of 0.1 to 0.5 psf, the force on the one inch square plate during the jet calibration must have been measured in thousandths of pounds. It would be of value to further investigators if the author would discuss briefly the difficulties experienced with this measurement and give the calibration curve obtained between "H" and the force on point "A" in Fig. 2. With such small forces, the reader wonders what influence the discontinuities between the steel Plate "A" and the base of the cylinder may have had on the water flow and forces measured. What were the clearances between steel Plate "A" and the base of the cylinder during the jet calibration?

The writer was not completely clear in his understanding of Table 3. What is the difference between the term "S_h" and the term "T_c" from Eqs. (11) or (21)?

REFERENCE

1. James L. Sherard, "Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams," a thesis submitted to Harvard University, March 1952.

COMPRESSIBILITY AS THE BASIS FOR SOIL BEARING VALUE[a]

Discussions by Michael Praszker and Morris Grosswirth

MICHAEL PRASZKER,[1] M. ASCE.—The author has re-arranged the basic equations relating to consolidation and has carried out a chain of algebraic substitutions that if employed by others should be handled with greatest care, and then only by those who are thoroughly familiar with the theory of consolidation.

Throughout the paper the author does not distinguish between fine textured, saturated materials and granular semi-saturated materials. The compression index C_c, as used by the author, appears to be a measure of compressibility for all soils. In present day analytical procedure the compression index C_c signifies the ratio $-\frac{de}{\log (dp)}$ or, in words, the gradient of the decrement in void ratio with relation to the increment of the logarithm of pressure when applied to fine textured, saturated materials which will undergo consolidation by virtue of gradually giving up the water from their voids upon the application of pressure increment. How the value of C_c can be obtained and interpreted in the case of dry or semi-saturated sands, the author has not attempted to explain. Compression in such sands need not necessarily be a result of consolidation. In the case of Fig. 3, the author does not mention whether these curves are based upon actual test results or whether they are hypothetical. It would be of interest to know how the author obtained the C_c and consequently $C = \frac{1 + e}{C_c}$ for sand and gravel.

Eq. (7), namely, $\Delta p = (10 \frac{SC}{1+} - 1)\, p_i$ which, in effect, is the key equation to the subject matter of the paper, will be discussed fully herewith. Moreover, since the term Δp in the above equation has not been defined by the author, it is here proposed to define this term, together with the other terms in the equation, in the light of the first principles of the Terzaghi theory of consolidation, namely:

Δp = the incremental pressure acting upon an elemental soil layer, or at the center of a thin soil layer, which undergoes consolidation by the action of the incremental pressure Δp. (It is generally considered preferable to express Δp in the form of a differential dp).

p_i = in situ pressure, or existing, or pre-existing pressure, in the elemental soil layer prior to the application of the incremental pressure Δp.

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a. Proc. Paper 2135, August, 1959, by C. K. Hough.

1. Partner, Lee & Praszker, Cons. Civ. Engrs., San Francisco, Calif.

S = ΔH = the amount of consolidation or reduction in thickness of the elemental layer which is acted upon by the incremental pressure Δp, and whose in situ, or existing, or pre-existing pressure, prior to the application of the incremental load was p_i.

$C = \frac{1 + e}{C_c}$ where

e = in situ void ratio, or existing, or pre-existing, void ratio in the elemental soil layer prior to the application of the incremental pressure Δp.

$C_c = \frac{d\,e}{d\,(\log p)}$ or, on a semi-log plot, C_c = the tangent to the straight line portion of the curve which relates the change in void ratio to the change in pressure.

H = the summation of all elemental soil layers undergoing consolidation.

With the terms thus defined, it is clear that the incremental pressure Δp and the resulting consolidation S in Eq. (7) apply solely to an elemental soil layer anywhere within a soil body without regard to the position of the elemental soil layer or to the position of the application of the incremental pressure. In the case of an infinitely loaded surface area the incremental pressures throughout the depth of the soil body are the same in every elemental horizontal layer, but in the case of a finitely loaded area the incremental pressures decrease with depth. If the total layer which undergoes consolidation is relatively thin then the pressure distribution within the thickness of the layer may be considered linear and the average incremental pressure may be taken as that at the center of the layer. In the case of a relatively thick soil stratum undergoing consolidation, the incremental pressure Δp will vary non-linearly with depth, and the error resulting from assuming straight line distribution may be considerable for a semi-finite loaded surface area.

It is now also clear that the relation between contact pressure (or surface loading) and the resulting incremental pressure induced in the underlying soil body cannot be expressed in algebraic terms unless the position of the compressible or consolidable layer with relation to the portion of the surface load is known. In the case of the consolidable stratum directly underlying the loaded surface, an algebraic expression can be obtained relating contact pressures to incremental pressures and resulting consolidation. A rational expression, mathematically speaking, in the case of a large loaded area (semi-finite area) and a thick consolidable soil layer, would be very laborious, since it would involve a process of integration.

Even with all the simplifying assumptions introduced by the author all expressions in the paper relate solely to one case, and one case only, namely, when the consolidable layer directly underlays the footing.

With regard to the term p_i, Eqs. (23), it is also pointed out that, even with all simplifications made in the equation, p_i need not at all be the initial stress due either to the pre-excavation surcharge or the existing surcharge. The initial stress p_i, in many instances, is much higher when dealing with preconsolidated clays.

Eq. (25), namely $p = \frac{\gamma}{27B^2}\left[(H + 3\,D)\,10\,\frac{S\,C}{H}\right](H + 3B)^2$ in which $H = h_s$ = significant depth, makes h_s or H (inter alia) dependent upon p_i. Closer

analysis of this equation shows that an assumption is here introduced that the thickness of the consolidable soil layer is at least equal to h_s.

In all cases, except the one stated above, the algebraic expressions presented by the author will not hold true, and recourse to the present day settlement analysis will be necessary. A few of the cases to which these algebraic expressions cannot be applied are illustrated on Diagram 1, Figs. 1 through 3.

The author's apprehensions (p. 32) to the effect that there exists an apparent contradiction between the compressibility theory and the rupture theory in designing foundation footings does not appear real. The balancing of footing sizes is now, and has been, practiced for a long time. In consolidable materials such as clays or silts, with negligible angle of internal friction, the ultimate bearing capacity does not increase with size of footings and there is no apparent contradiction. In the case of partly compressible sands the ultimate bearing capacity is obtained to determine merely a guide as to the minimum size footing required. The maximum safe soil pressures are then limited for the smallest footing size and the larger footings have their pressures reduced in such a fashion that the product of soil pressure ratios and their seats of settlement depths are approximately the same for all footings (seat of settlement may be defined as that depth within which 90% of settlement takes place due to an impressed net incremental pressure). This procedure is relatively simple and does not require algebraic expressions. The simplicity of this procedure permits the analyzer to view the position of the footing with relation to the position of the compressible layers, whether they may be directly under the footing or sandwiched between incompressible layers at some depth. An algebraic expression for such cases is difficult, if not impossible, to obtain, nor is it warranted.

In conclusion, the writer wishes to draw attention to the fact that algebraic expressions such as the author presents in his paper have the disadvantage of losing the reader, or the learning soils engineer, in a maze of direct and reciprocal expressions whose definitions take on a purely mathematical meaning. There is then the danger that a reader might take recourse to such curves as Diagram 5 without due regard to the specific condition of the foundations under study. Working with simpler equations and doing the algebra as the analysis progresses has obvious advantages.

MORRIS GROSSWIRTH,[1] M. ASCE.—Professor Hough should be commended for a most intelligent treatment of an important and practical topic. The writer concurs with the author's conclusions with the exception of the 3 points indicated below.

(1) In referring to Fig. 8(a) the author states . . . "that in so far as limitation of contact pressure to provide security against soil rupture is required at all it is needed only in determining the size of the smallest of a group of footings. All other footings should then be proportioned for equal settlement due to one dimensional soil compression, not shear". Fig. 8(a) refers to cohesive soils in general, with no distinction made between stiff, precompressed clays and soft, normally clays. Fig. 8(a) and the statement quoted above would more than likely be valid for the soft, normally loaded clays. That is, one might expect, for most size

1. Associate Prof., Dept. of Civ. Eng., New York University, University Heights, N. Y.

DIAGRAM 1.

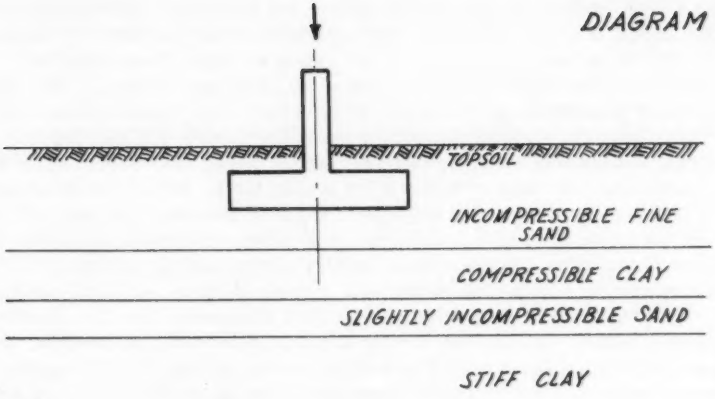

Fig 1.

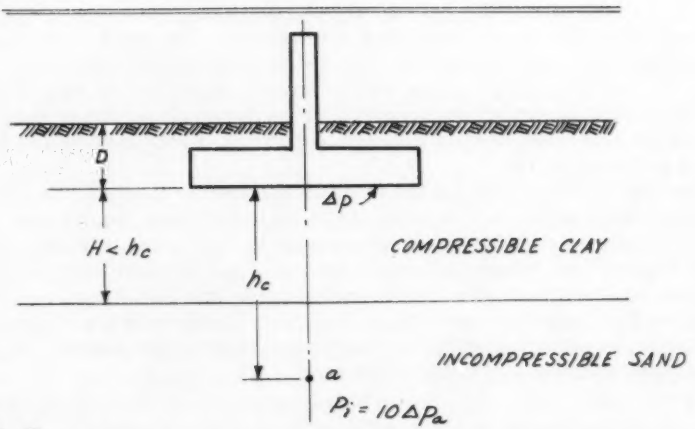

Fig 2.

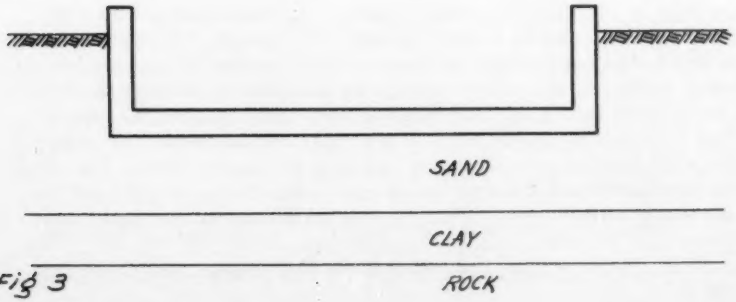

Fig 3

footings, that intolerable settlements would develop under contact pressures which were less than one-third of the ultimate bearing value of the underlying soil. However, for stiff, precompressed, it is more than likely, that settlements will not be excessive if the contact pressure does not exceed one-third of the ultimate value, in which case the shear rupture will control the magnitude of the allowable contact pressure. For this condition Fig. 8(a) should show all of the "S" curves (assuming that S represents some reasonable value, say 1 inch) as dashed lines located above the horizontal line labeled "P_{ult}/S.F.

(2) The author concludes that "the total load on surcharged footings should be less than that on similar footings without surcharge". He goes on to say that this conclusion is independent of the soil type. Even assuming that settlement controls, one would expect that at least for footings resting on granular soils a higher contact pressure would be necessary for surcharged footings in order to produce the same settlement under a non-surcharged footings. The surcharge would have the effect of increasing the confinement of the soil mass under the footing and as such increase its capacity to resist deformation, similar to the phenomenon exhibited in a triaxial test where for a given vertical stress, the vertical deformation produced will vary inversely with the chamber (confining) pressure.

(3) The author suggests that contact pressures be varied inversely with footing sizes in order to produce approximately the same settlements under all footings. While this is certainly desirable, there is some question as to whether this refinement is justified since it would be inconsistent with our ability to ascertain precise soil properties, which, at best, represent a scattering of values at a limited number of locations and depths. For most cases, it is usually sufficient to define the allowable contact pressure in terms of the heaviest column footing in combination with the average minimum soil conditions (of the entire site) which produce a settlement (assuming that shear rupture does not control) of some allowable magnitude (say 1 inch). All other footings should then be proportioned using this same contact pressure, in which case, even the lightest column footing will rarely settle less than 1/2 inch and in no case would be less than 3/4 of an inch so that differential settlements between the heaviest and lightest column footing will in all likelihood be less than 1/2 of an inch and never larger than 3/4 of an inch.

Professor Hough deserves thanks for making this significant contribution on a subject which is marked by a lack of agreement among soils engineers and one for which many Building Codes flagrantly ignore the latest and most scientific knowledge available.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST8), 1866(ST8), 1867(ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876(WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926(PL1), 1927(HW), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1),1956(CO1),1957(CO1), 1958(CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968(SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977(WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998(SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007(HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

MAY: 2014(AT2), 2015(AT2), 2016(AT2), 2017(HY5), 2018(HY5), 2019(HY5), 2020(HY5), 2021(HY5), 2022(HY5), 2023(PL2), 2024(PL2), 2025(PL2), 2026(PP1), 2027(PP1), 2028(PP1), 2029(PP1), 2030(SA3), 2031(SA3), 2032(SA3), 2033(SA3), 2034(ST5), 2035(ST5), 2036(ST5), 2037(ST5), 2038(PL2), 2039(PL2), 2040(AT2)[c], 2041(PL2)[c], 2042(PP1)[c], 2043(ST5)[c], 2044(SA3)[c], 2045(HY5)[c], 2046(PP1), 2047(PP1).

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JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087(SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096(EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3),2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3),2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3),2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

AUGUST: 2126(HY8), 2127(HY8), 2128(HY8), 2129(HY8), 2130(PO4), 2131(PO4), 2132(PO4), 2133(PO4),2134(SM4), 2135(SM4), 2136(SM4), 2137(SM4), 2138(HY8)[c], 2139(PO4)[c] 2140(SM4)[c].

SEPTEMBER: 2141(CO2),2142(CO2),2143(CO2),2144(HW3),2145(HW3), 2146(HW3), 2147(HY9), 2148(HY9), 2149(HY9),2150(HY9),2151(IR3), 2152(ST7)[c], 2153(IR3), 2154(IR3), 2155(IR3), 2156(IR3), 2157(IR3), 2158(IR3),2159(IR3),2160(IR3),2161(SA5),2162(SA5),2163(ST7),2164(ST7), 2165(SU1), 2166(SU1),2167(WW3), 2168(WW3), 2169(WW3), 2170(WW3), 2171(WW3), 2172(WW3), 2173(WW3), 2174(WW3), 2175(WW3), 2176(WW3), 2177(WW3), 2178(CO2)[c], 2179(IR3)[c], 2180(HW3)[c], 2181(SA5)[c], 2182(HY9)[c], 2183(SU1)[c], 2184(WW3)[c], 2185(PP2)[c], 2186(ST7)[c], 2187(PP2), 2188(PP2).

OCTOBER: 2189(AT4), 2190(AT4), 2191(AT4), 2192(AT4), 2193(AT4), 2194(EM4), 2195(EM4), 2196(EM4), 2197(EM4), 2198(EM4), 2199(EM4), 2200(HY10), 2201(HY10), 2202(HY10), 2203(PL3), 2204(PL3), 2205(PL3), 2206(PO5), 2207(PO5), 2208(PO5), 2209(PO5), 2210(SM5), 2211(SM5), 2212(SM5), 2213(SM5), 2214(SM5), 2215(SM5), 2216(SM5), 2217(SM5), 2218(ST8), 2219(ST8), 2220(EM4), 2221(ST8), 2222(ST8), 2223(ST8), 2224(HY10), 2225(HY10), 2226(PO5), 2227(PO5), 2228(PO5), 2229(ST8), 2230(EM4), 2231(EM4), 2232(AT4)[c], 2233(PL3)[c], 2234(EM4)[c], 2235(HY10)[c], 2236(SM5)[c], 2237(ST8)[c], 2238(PO5)[c], 2239(ST8), 2240(PL3).

NOVEMBER: 2241(HY11), 2242(HY11), 2243(HY11), 2244(HY11), 2245(HY11), 2246(SA6), 2247(SA6), 2248(SA6), 2249(SA6), 2250(SA6), 2251(SA6), 2252(SA6), 2253(SA6), 2254(SA6), 2255(SA6), 2256(ST9), 2257(ST9), 2258(ST9), 2259(ST9), 2260(HY11), 2261(ST9)[c], 2262(ST9), 2263(HY11), 2264(ST9), 2265(HY11), 2266(SA6), 2267(SA6), 2268(SA6), 2269(HY11)[c], 2270(ST9).

DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278(HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287(PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296(SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322(ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330(CP2).

[c] Discussion of several papers, grouped by divisions.

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SM 6

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Your attention is invited

NEWS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION OF ASCE

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES
SOIL MECHANICS AND FOUNDATIONS DIVISION
Proceedings of the American Society of Civil Engineers

NEWS

December, 1959

FIFTH INTERNATIONAL CONFERENCE ON SOIL MECHANICS AND FOUNDATION ENGINEERING PARIS 17 - 22 JULY 1961

The Organizing Committee for the Fifth Conference has sent a copy of their draft of Bulletin No. 1 for the Conference to each of the National Committees of the International Society. With their forwarding letter they state that printed copies of the Bulletin will be sent out in September 1959. (As of 23 October, copies have not yet been received.) Your Secretary has requested that copies of this Bulletin go to each member of the United States National Committee and that an additional 50 copies be sent to the Secretary's office for distribution to other interested parties.

In brief, the draft of Bulletin No. 1 outlines the organization of the Conference to be held in Paris, gives instructions to authors for preparation of papers and furnishes a preliminary application form for attendance. Following is a resume of certain information in the Bulletin which is believed to be of general interest.

July 1961

- 9	Large Dams Congress, Rome - International Commission on Large Dams
	Fifth International Congress of Soil Mechanics and Foundation Engineering
10 - 14	a) Study tour south-east France, starting point Nice b) Study tour south-east France, starting point Paris
17 - 22	Fifth International Congress on Soil Mechanics and Foundation Engineering - Paris
24 - 30	c) Study tour French Alps, Basing of the Durance and Rhone Rivers, starting point Paris
24 - 27	d) Study tour Basin of the Seine, Dunkirk, starting point Paris

The fee for registration at the Conference will be about $51 for delegates

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and about $21 for ladies. The $51 includes the cost of the three volumes of Proceedings of the Conference. Additional sets of Proceedings may be procured by National Committee members at a cost of approximately $25. The first two volumes will be published in advance of the Conference and will contain the papers and the general reports. The third volume will record the discussions of papers, visits and daily affairs of the Conference and will be published about six months after the Conference.

The papers of the Conference will be divided into the same seven categories as used for the Fourth Conference in London. Discussions at Technical Sessions will be led by a chairman and will be initiated by the general reporter for the particular session. Discussions will be by specialists designated in advance who have made a special study of the subject in question and by certain authors of papers and of written discussions. If time permits, the chairman will open the discussion to the general assembly. Discussions will be directed toward the following particular items:

Section I (Soil properties and their measurement).

a) Scatter of soil mechanics tests
b) Pore-pressure dissipation
c) Visco-elastic properties of soils; the existence of a Bingham limit; flow

Section 2 (Techniques of field measurement and sampling).

In-situ tests for determination of mechanical properties of soils. Comparison with laboratory results.

Section 3 (Foundations of structures).

A - Shallow Foundations

a) Influences of the shape and the dimensions of the foundations.
b) Non-saturated soils. Expansion and shrinkage.

B - Piled Foundations

a) Determination of the bearing capacity of a foundation from penetrometer tests.
b) Pile groups; bearing capacity and settlement.

Section 4 (Roads, runways, and rail-tracks).

Deformation of roadways.

Section 5 (Earth Pressure on structures and tunnels).

Variation in the active and passive pressure related to the deformations in the vicinity of the structure.

Section 6 (Earth dams; slopes, and open excavations)

Stability of slopes: considered as a function of time; effect of engineering works.

Section 7 (Various questions)

Best laboratory-works relationship. Use of soils as a constructional material in housing projects.

Regarding papers from the United States, about fifty-five summaries of proposed papers have been received. Unfortunately, as in the case of the

Fourth Conference at London, space for only twenty-five papers has been allocated to the U.S.A. by the Organizing Committee. Authors of proposed papers will be advised of the requirements for submission of their manuscripts and of the status of their papers as soon as such information is available.

Bulletin No. 2 of the Conference will give full details of the programs, fees, tours and excursions. This Bulletin is scheduled to be sent out by the French Organizing Committee in December 1960. It will be sent directly to members who have sent in the preliminary application form contained in Bulletin No. 1.

THIRTEENTH CANADIAN SOIL MECHANICS CONFERENCE

Earth dams, both new and old, were the central topic of discussion at the Thirteenth Canadian Soil Mechanics Conference. The old type of earth dams built in Nova Scotia over 200 years ago were described. These were built by early settlers in their attempts to keep tidal water out of the valuable agricultural land, the dams being necessary because of the very high tides experienced in the Bay of Fundy. Some of these old dams, locally called aboiteaux, were of appreciable size and many have been in daily use successfully for over 200 years. Modern dams for performing the same function were also described and a review given of the construction of the many small earth dams on the rivers of Nova Scotia that are now in use for the creation of reservoirs for the many small water power plants which serve that province.

The Conference was held on 10 and 11 September 1959 at the Nova Scotia Technical College in Halifax by invitation of the President, Professor J. Hoogstraten. Dr. G. G. Heyerhof, Head of the Department of Civil Engineering at the College, acted as Chairman of the local committee, which was responsible for the arrangements for the Conference.

Speakers from Prince Edward Island, Nova Scotia, and New Brunswick presented papers on the first day to about 120 engineers and scientists gathered from all parts of Canada. All the provinces of Canada were represented and most of the Canadian universities at which soil mechanics forms a part of the civil engineering curriculum.

Professor H. L. Cameron, of Acadia University, Nova Scotia, emphasized in the opening paper the influence geology has on engineering in the Maritime provinces, particularly the importance of geology in locating desirable materials for engineering structures. He suggested that geology can best contribute in two ways to engineering construction: can we build what we require on the site selected and what materials can the area contribute to construction needs?

Mr. J. D. Conlon, in describing the Annapolis River Dam and Highway Causeway at Annapolis Royal, said the dam will also function as an aboiteau in allowing the discharge of fresh water from the Annapolis River and preventing the return flow of salt tidal water under the influence of the local 40-ft. tide. This structure will protect 4300 acres of arable marshland above the site.

Earth dam construction in Nova Scotia in connection with power development was described by Messrs, G. J. Currie, A. MacDonald and K. Whitman, who pointed out that although the hydro potential was not great in Nova Scotia, the use of earth dams would play an increasingly important part in its development. Ninety earth dams have been constructed to date for this purpose.

The chief soil problems during construction of the Beechwood Power Project in New Brunswick were excessive soil moisture and wet weather. Professor H. W. McFarlane, of the University of New Brunswick, stated that the most unusual feature was the use of an asphalt aggregate plant as a soil drier; he showed that in this particular case it had proved to be an effective and economical expedient.

During the second day of the Conference, more general aspects of soil mechanics in Canada were presented. Mr. C. O. Brawner, of the B. C. Department of Highways, reported on recent landslides in British Columbia. He stated that each year $2 million is spent to stabilize or prevent earth movements in critical locations. The volume of material involved in slides had been as high as 20 million cubic yards, as in the case of Drynoch Slide. He described the present remedial measures used, the most common being counter-balancing and subsurface drainage when relocation of highways was not possible.

Mr. C. B. Crawford, of the Division of Building Research, described the engineering properties of Leda clay, a soft marine clay from the St. Lawrence and Ottawa Valleys, and the research being carried out in the N.R.C., D.B.R., Soil Mechanics Section. He pointed out that the recent trend had been to concentrate on strength properties of clays. An attempt was being made to assess the strength properties of clays in terms of effective stresses; much time had been spent in studying time effects and in accurately measuring pore water pressures during shearing. Much attention had also been recently to the validity of the standard consolidation test.

Repetitive loading tests on undisturbed clays was discussed by Professor V. E. Vaughan, of the Nova Scotia Technical College, who described work done in assessing the changes in strength characteristics resulting from different loading histories. The final presentation was a paper by Mr. N. D. Lea, of Vancouver, who discussed the proposed Canadian Standards Association specifications for boring and sampling which was the result of the work of a CSA committee of which Mr. Lea is chairman. The discussion that followed brought out the diversity of opinions on this subject but it was obvious that much headway had been made during the past few years.

During the Conference a dinner meeting was sponsored jointly by the Association of Professional Engineers of Nova Scotia and the Halifax Branch of the Engineering Institute of Canada, at which the guest speaker was Mr. R. Peterson of the Prairie Farm Rehabilitation Administration, Saskatoon, who spoke on the construction of the South Saskatchewan Dam.

The annual Canadian Soil Mechanics Conferences are sponsored by the National Research Council through its Associate Committee on Soil and Snow Mechanics which serves as the national committee for Canada of the International Society of Soil Mechanics and Foundation Engineering. Plans for the conferences are developed by the Soil Mechanics Subcommittee with the assistance of local groups, such as that headed by Dr. Meyerhof for the Halifax meeting. The proceedings of the Halifax conference will be published shortly as a Technical Memorandum by the Associate Committee. Copies may be obtained from the Secretary, Associate Committee on Soil and Snow Mechanics, c/o the Division of Building Research, National Research Council, Ottawa.

COLORADO SECTION

The activities of the Soil Mechanics and Foundations Division, Colorado Section, for the 1958-59 year are shown in the "Summary of Meetings."

October 1958 to June 1959
Soil Mechanics and Foundation Division
Colorado Section
American Society of Civil Engineers

All meetings were held in the Stearns-Rogers Building, Denver.

NO.	DATE	SPEAKER	TITLE	ATTENDANCE
1	October 1, 1958	Mr. Norman D. Baumgart Engineer Portland Cement Assn. Denver, Colorado	"Some Engineering Aspects of Soil-Cement Mixes"	13
2	November 5, 1958	Mr. Eugene Waggoner Consulting Geologist Denver, Colorado	"New Subsurface Eyes and Hands for the Geologist and Engineer"	26
3	December 3, 1958	Mr. W. D. Farrington Civil Engineer Woodward-Clyde Sherard & Associates Denver, Colorado	"Foundation Failures"	14
4	January 7, 1959	Mr. Harold E. Eyrich Manager, Materials & Substructures Denver, Colorado	"Soils and Soils Cement, their Treatment and Uses"	14
5	February 4, 1959	Mr. William E. Collins Asst. Section Chief of Dam Section U.S.B.R.	"Dam Construction in Japan"	16
6	March 4, 1959	Mr. Harold Gibbs U. S. Bureau of Information Denver, Colorado	"The In Place Vane Shear Test"	14
7	April 1, 1959	Mr. Fu Hua Chen Civil Engineer Woodward-Clyde-Sherard & Associates Denver, Colorado	"Soil Conditions and Foundations Problems in Hong Kong"	22
8	May 6, 1959	Mr. E. Vern Konkel Ketchum, Konkel & Hastings Denver, Colorado	"Structural Engineers Viewpoint on Soil Engineers and Professional Relations between these two Branches of Civil Engineering"	21
9	June 3, 1959	Mr. David J. Varnes U.S.G.S.	"Landslides and Engineering Problems"	17

In addition to the monthly meetings indicated, the local section co-sponsored with the Civil Engineering Department of the Colorado School of Mines a one-day conference on the Theoretical and Practical Treatment of Expansive Soils. This conference was highly successful and over 200 individuals registered and attended the conference.

The program of this one-day conference was as follows:

Welcome Address

John W. Vanderwilt, President, Colorado School of Mines
Leo Novak, President of Colorado Section, ASCE
T. A. Kelly, Head, Civil Engineering Department, Colorado School of Mines

"Building on Expansive Clay"

Professor R. E. Means, Professor of Architecture and Civil Engineering, Oklahoma State University

"The Role of Effective Stress
in the Behavior of Expansive Clays"

Dr. T. William Lambe and Professor R. V. Whitman, Department of Civil Engineering and Sanitation, Massachusetts Institute of Technology

"Expansive Clays,
Properties and Problems"

Mr. W. G. Holtz, Division of Engineering Laboratories, Denver Federal Center, U. S. Bureau of Reclamation

"Modern Practices Used in the Design
of Foundations for Structures on Expansive Clays"

Mr. Raymond F. Dawson, Professor of Civil Engineering and Associate Director, Bureau of Research, University of Texas

"The Relations of Laboratory Testing
to Design for Pavements and Structures on Expansive Soils"

Mr. Chester McDowell, Senior Soils Engineer, Texas Highway Department

Panel Discussions

Inquiries on the proceedings have been received from over 600 additional people. The proceedings are being published by the Colorado School of Mines for release in October.

In addition to a program planned for next March at the Colorado School of Mines, the Colorado Section will host a Research Conference on Shear Strength in June of 1960.

During the June 1959 meeting the following men were elected officers for 1959-60.

Wilmer W. Daehn, Chairman
Kalman Zeff, Vice-Chairman
Claude C. Klemme, Secretary

C/of Commercial Testing Laboratories
22 Lipan
Denver 23, Colorado

M.I.T. PROGRAM ON SHEAR STRENGTH OF SOILS

The Soil Engineering Division of the M.I.T., Department of Civil and Sanitary Engineering, completed on September 11, 1959, its two week summer program on "The Shear Strength of Soil." The program was attended by 90 registrants, of whom 73 came from 30 states and 17 from five foreign countries.

The program, which was aimed at providing practicing engineers, researchers, and educators with an up-to-date understanding of the fundamental nature and use of shear strength in engineering problems, consisted of about 3 hours of lecture and discussion each morning and afternoon and several evening sessions. A tour of the M.I.T. Soil Engineering Laboratory and

demonstration of the latest equipment for triaxial testing with pore pressure measurements was also included.

Dr. T. William Lambe, Professor and Head of the Soil Engineering Division, directed the program and talked on soil technology and its application to problems involving clays. Dr. Laurits Bjerrum, Director of the Norwegian Geotechnical Institute and Visiting Professor at M.I.T., lectured for two days on the use of the effective stress principle in solving various categories of stability problems with different soil types. Dr. A. W. Skempton, Professor and Head of Civil Engineering, Imperial College of Science and Technology, University of London, discussed for one and one-half days his experiences with cuts, slopes, foundations, etc. in stiff-fissured clays and the magnitude of pore pressures in compacted earth dams. Dr. R. V. Whitman, Associate Professor of Soil Engineering at M.I.T., lectured on stability analyses using drained and undrained strength and on the influence of strain-rate on strength. James E. Roberts, Assistant Professor of Soil Engineering, discussed the strength of partially saturated soils in terms of effective stress and the use of shear strength in the design of footings and driven pile foundations. Other topics, such as the laboratory measurement of shear strength, the use of strength data in the design of flexible pavements and the strength concepts of Hvorslev, Henkel and Roscoe, were presented by Messrs, C. C. Ladd, R. M. Harkness and A. M. Richardson of the M.I.T. Staff.

The discussion periods which accompanied the lectures were very stimulating and instructive due to the extensive training and experience of the registrants.

All was not work, however, as two cocktail parties and a banquet were well attended. Dr. Karl Terzaghi, Professor of the Practice of Civil Engineering, Emeritus, Harvard University and Lecturer at M.I.T., was the guest speaker at the banquet. His topic: examples of the instability of slopes due to unusually high pore pressures.

Since the program consisted of a series of informal lectures and discussion periods, there were no formal handouts, except the following:

"Notes on the Basic Principles of Shear strength" and "Slope Stability Analysis," both by Professor Whitman. These may be purchased for $5,00 and $3.00 respectively by writing the Soil Engineering Division.

NEWS FROM THE WEST COAST

Visit of Russian Soils Engineers to the University of California

During the week of June 14 the delegation of Russian Soil Mechanics Engineers visited the San Francisco Bay Area. On Monday, June 15 an all-day seminar was held on the Berkeley campus of the University of California on problem of soil mechanics and foundation engineering of mutual interest. Topics discussed were grouped under two general headings:

1. Design of Foundations and Earth Structures
2. Field Exploration, Foundation Construction and Miscellaneous Subjects.

That evening the Soil Mechanics and Foundation Division entertained the group at their regular meeting. The Russians presented their film on Vibratory pile driving.

On June 16 the group was entertained in Sacramento by a tour through the new Materials and Research Laboratory of the Division of Highways, State of California.

Meetings

On June 15 a meeting was held at the University of California in Berkeley by the Soil Mechanics and Foundation Division. The technical program was presented jointly by the Russian Soils Mechanics Engineers and the local Section with more than 100 in attendance. In addition to the Russian movie on "Vibratory Methods of Pile Driving" the following short talks were presented by members of the Society:

"Land Subsidence Problems in California" by J. F. Poland.

"Foundations for the Kaiser Building, Oakland and Zellerbach Building, San Francisco" by W. W. Moore

"Earthquake Damage" by Karl V. Steinbrugge

On August 4 a meeting was held by the Division in Orinda, California during which a short lecture was presented on use of "Nuclear-Chicago" - A/M surface units for determination of moisture and density of soils in situ. Following, the lecture and field demonstration of the equipment was presented at the proposed Briones Dam Site located just east of Berkeley, California.

The dam is presently under design by the East Bay Municipal Utility District and in connection with the soils investigation test embankments are being constructed to determine the most efficient method of excavating and compacting the borrow materials. The demonstration of the nuclear equipment was made in connection with the standard determination of moisture and density using the sand cone and "Washington Densometer" techniques.

On the 14th of October the Soil Mechanics and Foundation Division of the San Francisco section held a meeting which was addressed by Mr. Stanley F. Gizienski, Consulting Civil Engineer of Woodward, Clyde, Sherard and Associates.

Mr. Gizienski presented a talk on the effects of the recent Montana Earthquake on an existing earth dam and the formation of a new natural dam. The earthquake shook the Madison River Valley just west of Yellowstone National Park on the night of August 17, 1959, creating large landslides which in turn caused a considerable loss of life among vacationers in the area. The earthquake caused a huge landslide to block the mouth of the Madison Canyon thereby creating a natural dam and flooding recreational areas in the Canyon. It also damaged the rock and earth Hebgen Dam located upstream about seven miles.

Potential failure of the Hebgen Dam and release of about 300,000 acre-feet of water over the landslide created the possibility of flood hazards to communities located in the Madison River Valley downstream; therefore the Governor of Montana requested that the U. S. Corps of Engineers investigate the situation. Mr. Gizienski and Dr. Sherard of Woodward, Clyde, Sherard's New York Office, at the request of the Corps of Engineers, made a detailed field reconnaissance of the dam and the slide to determine their safety and made recommendations to the Corps. The recommendations are now being carried out in the field.

Mr. Gizienski illustrated his talk with colored slides which he took from helicopters flying over the sites.

During the week of October 11 to 16, 1959, the ASTM held its Third Pacific Area National Meeting in San Francisco. On Friday, October 16, a symposium was presented as part of this meeting on "Soils for Engineering Purposes." The program presented consisted of the following:

Preprint Number	
15	Measurement of Excess Hydrostatic Pressures in Soils — W. G. Weber, Jr., California Division of Highways.
16	Moisture and Strength Variation in a Thick, Uniform Clay Layer — W. G. Weber, Jr., and W. F. Kleiman, California Division of Highways.
17	Extended Repetitive Plate Load Tests for the Evaluation and Design of Concrete Pavements — B. B. Gordon, Porter, Urquhart, McCreary & O'Brien.
18	Some Laboratory Studies of the Moisture-Density Relations of Soils — R. F. Dawson, University of Texas
19	On Determination of Dynamic Characteristics of Soils in situ — R. H. Bernhard, Rutgers State University.
20	The Effect of Temperature on Moisture Contents as Determined by Centrifuge and Tension Techniques — R. C. Prill and A. I. Johnson, U. S. Geological Survey.
21	The Effect of Particle Shape and Texture on the Strength of Non-Cohesive Aggregates — H. C. Morris, University of Washington.
22	Triaxial Compression Tests on Soils Using Variable Lateral Pressure — R. H. Meese and R. W. Long, University of Washington.
23	The Development of A Soils Classification Triangle Based on the Unified Soil Classification System — Jack McMinn, Brewer and Associates.

Advance prints of some of these talks are available from ASTM Headquarters. It is expected that some time after the meeting most of these papers will be published as part of the ASTM Special Technical Publication Series.

I. M. LITVINOV ADDRESSES NATIONAL ACADEMY OF SCIENCES

One June 18, 1959, the National Academy of Sciences was host in Washington, D. C. to the Soviet Delegation of Specialists on Soil Mechanics and Foundation Engineering. On this occasion, I. M. Litvinov, Academician-Secretary. Presidium, Academy of Construction and Architecture, Ukrainian SSR was the featured speaker.

Mr. Litvinov was effusive in his praise of what he and his Soviet colleagues had seen during their visit of the United States. Their visits took them to many universities and to several of the laboratories operated by branches of the federal government. The seminars that were held in conjunction with these visits provided an opportunity to exchange ideas and techniques.

The Soviet engineers were well-informed regarding American technology as a result of the wide publication programs that they have undertaken. These publications keep them aware of the contributions of American engineers. Of particular interest, was "the remarkable organization of the test road research. . . . The considerable achievements of American specialists in the field of road construction are generally acknowledged. In this respect, the American roads are presentations in nature equivalent to beautiful books and scientific studies."

Mr. Litvinov outlined the steps followed to create the Academy of Construction and Architecture of the USSR and the Ukranian Academy of Construction and Architecture. These two academies have more than 12,000 workers engaged in the solution problems in the fields of construction and architecture.

Mr. Litvinov stated that the most often-heard question was what they liked best about America. They would answer: "The Americans who, just as we do, want a peaceful life and friendly exchanges in business and scientific fields." And if they were asked what they liked least they would respond: "The short days, since we would have wished that during our stay in America a day had 48 hours and not 24. Then we could have seen and learned even more of interest to us."

Mr. Litvinov thanked the National Academy of Sciences and ASCE for the initiative and efforts required to bring about their trip to the United States. Special thanks were given to Professor Gregory P. Tschebotarioff.

ROCK MECHANICS SYMPOSIUM

Golden, Colorado - - - The proceedings of the Third Annual Rock Mechanics Symposium, held at the Colorado School of Mines, April 20 through 22, have been published as a CSM Quarterly.

The Symposium, supported by a $2500 National Science Foundation grant this year, was co-sponsored by Mines, the University of Minnesota and Pennsylvania State University. More than 300 industrial and research engineers and educators attended the conference and heard papers presented by outstanding engineers and scientists from five nations.

The Symposium was divided into four major sections, each dealing with a distinct area of rock mechanics. They treated nuclear blasting, geophysics, soil and rock mechanics factors, and common underground and explosive failures.

Although present day connections of rock mechanics to the mineral industries is largely scientific, the authors state this linkage will become a practical relationship and application in the future.

Included among the papers published in the Quarterly are those of Dr. Leopold Muller, President of the International Association of Geomechanics, Germany; L. E. Djingheusian, senior Canadian Department of Mines engineer; Ulf Langefors, Nitrolglycerin Aktiebolaget physicist, Sweden; Sir Harold Jeffreys, Plumian Professor of Astronomy and Experimental Philosophy, Cambridge University, England; Dr. John S. Rinehart, head of the CSM

mining engineering research laboratories; and a nuclear blast summation by four staff engineers of the Lawrence Radiation Laboratory of the University of California.

The 370-page publication, Volume 54, number 3, is priced at $3.00. It is available from the Department of Publications, the Colorado School of Mines, Golden, Colorado.

SOIL SCIENCE

Mr. E. F. Williams, President of the Williams and Wilkins Company publishers of books and periodicals in medicine and the allied sciences has called to our attention the journal SOIL SCIENCE. This journal is published under the editorial direction of the department of soils of Rutgers University. It contains a variety of research papers which may be of interest to some members of this division.

The first issue of SOIL SCIENCE was published in 1916. It is now in its 88th volume. During the last 12 months it published 111 papers of which 37 were from foreign countries. It has something over 2600 subscribers of which more than two-thirds live in foreign lands.

The subscription price to SOIL SCIENCE is $5.00 per volume of 6 monthly issues of $10.00 per year. Subscriptions should be sent to The Williams and Wilkins Company, 428 East Preston Street, Baltimore 2, Maryland. Research papers offered for publication; which is free, should be sent to Dr. Firman E. Bear, Rutgers University, New Brunswick, New Jersey.

FEBRUARY NEWSLETTER

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